APPENDIX B

Groundwater Studies, Including Drinking Water Well Inventory and Groundwater Mounding Analysis

Pacific Groundwater Group

GROUNDWATER INVESTIGATION AND MOUNDING ANALYSIS IN SUPPORT OF THE TULALIP TRIBES' TREATED WASTEWATER INFILTRATION SYSTEM

Prepared by:
Pacific Groundwater Group
2377 Eastlake Ave. East
Seattle, WA 98102

Prepared for:
Parametrix, Inc.
PO Box 460
Sumner, WA 98390

May 15, 2002

JE0108

Table of Contents

List of Tables

List of Figures

Signature

This report, and Pacific Groundwater Group's work contributing to this report, were reviewed by the undersigned:

Charles T. Ellingson
Washington State Licensed Hydrogeologist No. 631

GROUNDWATER INVESTIGATION AND MOUNDING ANALYSIS IN SUPPORT OF THE TULALIP TRIBES' TREATED WASTEWATER INFILTRATION SYSTEM

This report summarizes work conducted to simulate infiltration of treated effluent from the Tulalip Tribes' commercial development west of Interstate 5 and north of the Snohomish River, between NE 88th and NE 116th, in Snohomish County. The following work was conducted by Pacific Groundwater Group in coordination with Parametrix' wastewater treatment and disposal planning, permitting, and design efforts:

- Participated in the design, execution, and analysis of infiltration tests.
- Evaluated a groundwater model (MODFLOW) of the area developed by Landau Associates.
- Developed a modeling plan to address the needs of this project.
- Simulated infiltration of treated effluent using a new MODFLOW model.
- Reported on the findings.

The work was authorized by Parametrix on November 11, 2001, under Parametrix contract 216-1598-012. Pacific Groundwater Group performed the work, and prepared this report, using hydrogeologic practices generally accepted in this area at this time, for exclusive use of Parametrix and specific application to the project. This is in lieu of other warranties, express or implied.

REGIONAL HYDROGEOLOGY

The site lies in the Marysville Trough, a three-to-four-mile-wide physiographic feature extending from the Snohomish River on the south to the Stillaguamish River on the north (**Figure 1**). The following geologic units are known to occur in the Marysville Trough and are described from shallowest to deepest (EES, 1991; Thomas, Wilkinson, and Embrey, 1997; Wert and Associates, 1995). **Table 1** provides additional information on lithology, unit thicknesses, and unit top elevations.

- Qvr Vashon Recessional Deposits (sand with silt layers)
- Qvt Vashon Till (cemented sand, silt, and gravel)
- Qva Vashon Advance Deposits (fine sand to gravel)
- Qtb Transition Beds (clay, silt, and fine sand)

Most carefully logged borings in the project vicinity indicate fine to medium sand below surface soils, then a thin silt or silty sand layer that typically lies 10 or 15 feet below ground surface within the upper portion of the Qvr deposit. Typically, the silt is underlain by fine to medium sand in the lower portions of the Qvr deposits.

Shannon and Wilson (1994) logged substantial silt interbedded with sand in the Qvr unit, beginning at about 40 feet depth, at the intersection of 88th and I-5. This information suggests that the lower limit of the Qvr unit may be as shallow as 40 feet below ground surface at that location, or that Qvr sands are divided into shallower and deeper portions, separated by silt beds. As noted in **Table 1**, both data and interpretations indicate variable thickness of the Qvr unit.

Groundwater Occurrence and Flow

A water table occurs within 20 feet of land surface in the project area. The saturated Qvr sands below the water table constitute the Marysville Trough Aquifer (MTA). The MTA is a well-documented regional hydrogeologic feature. Groundwater in the aquifer in the project vicinity flows generally southeasterly. Groundwater flows from the Tulalip Plateau and the margins of the Trough on the west, to Quil Ceda Creek on the east, to the smaller Coho and Sturgeon Creeks on the southeast, and to the Snohomish River south of the project (**Figures 1 and 2**).

Upward flow from the Qva to Qvr in the Marysville Trough was reported by the USGS (Thomas, Wilkinson and Embrey, 1997); and Landau Assciates (1999) reported a flowing artesian well at the western margin of the Trough near the site. Thus, the MTA also receives recharge from deeper hydrogeologic units. Much of the trough appears to be a discharge area for the regional groundwater flow from the uplands to the west and east. The resulting upward flow, gentle slope from the edges of the trough to Quil Ceda Creek, and shallow incision of the underfit creek relative to the broad floodplain of the former glacial meltwater river, all contribute to the shallow depth to groundwater.

Estimates of the hydraulic properties of the Marysville Trough aquifer (MTA) are available in several reports. EES (1991) estimated that the regional transmissivity ranges from 1,336 ft2/day to 6,684 ft2/day, and that the horizontal hydraulic conductivity ranges from 50 to 200 ft/day. The estimates generated by Wert and Associates, Landau, and AGRA discussed below are all for the project area. Wert and Associates (1995) estimated that vertical hydraulic conductivity of near-surface sands is approximately at 7 ft/day. Landau Associates (1999) estimated that non-directional hydraulic conductivity ranges from 53 ft/day to 140 ft/day, based on grain-size analyses. AGRA (1996) estimated that transmissivity averages 2,673 ft2/day, horizontal hydraulic conductivity averages 85 ft/day for an aquifer thickness of 31 feet, and specific yield averages 0.03, based on a 24-hour aquifer pumping test.

Short-term aquifer tests using wells P-1 through P-9 (**Figure 2**) were performed by AMEC for this project (**Table 2**). Data from those tests indicate minimum, median, and maximum hydraulic conductivities of 13, 47, and 156 ft/day when analyzed without correction for partial penetration.

Landau Associates (1999) measured water levels in several MTA monitoring wells beginning in early 1999. These measurements were supplemented by recent data collected for this project. **Table 3** presents the recent data and **Figure 3** presents hydrographs for selected wells with the longest records (early data from **Figure 3** are not listed in **Table 3**). The hydrographs indicate that shallow groundwater levels rise to a maximum early in the year and recede to a minimum in early fall. The range of annual fluctuation ranges from about 4 feet to 6 feet, depending on location. Maximum fluctuations are expected in areas furthest from drainage features (ditches and streams) and areas of groundwater flooding (the west side).

The maximum groundwater elevations measured in the numerous piezometers through the winter of 2001-2002 (**Table 3**) are not as high as historical maximum groundwater elevations based on the long-term hydrographs shown in **Figure 3**. The long-term precipitation record (**Figure 4**) also indicates that shallow groundwater levels in the winter of 2001-2002 are not likely as high as the historical maximum. This fact was considered in the assessment of feasibility of treated effluent infiltration.

The maximum groundwater elevations measured in December 2001 through January 2002 were plotted to assess groundwater flow directions during this period of high groundwater levels (**Figure 2**). Elevations of Coho and Quil Ceda Creeks were also surveyed and considered in the contouring of **Figure 2**. Elevations in this report are all to the NGVD29 datum which is used exclusively by the Tribe. Data collected by Landau Associates were translated from the NAVD88 datum by subtracting 3.71 feet.

Beneficial Use of Local Groundwater

Wells pump groundwater from the MTA for the typical variety of private and public beneficial uses. Well yields from the MTA range up to 300 gpm.

Water supply wells within 0.25-mile of the infiltration trenches occur east of Interstate 5 (**Figure 5**). The locations mapped are approximate and dependent on reported locations. Also, undocumented wells may exist. Records of fourteen water supply wells occur in the 0.25-mile radius area. In addition, two wells exist just east of the area and west of Quil Ceda Creek. The wells were identified based on review of the following prior publications and records:

- Beneficial Water Use Survey, Tulalip Test Site, Marysville WA. August 3 2000. Landau Associates. Memorandum Prepared for the Boeing Company
- The Ground-Water System and Ground-Water Quality in Western Snohomish County Washington. B.E. Thomas, J.M. Wilkinson, and S. S. Embrey, USGS Water-Resources Investigation Report 96-4312
- Water Resources of the Tulalip Indian Reservation, Washington. 1983. USGS Water-Resources Investigations Open-File Report 82-648

- Water Well logs from Washington State Department of Ecology
- Water Right Tracking System of Washington State Department of Ecology
- Locations of water supply sources for public water systems compiled by Washington State Departments of Transportation and Health.

The concentration of water supply wells in the northwest corner of Section 21 is based on the Beneficial Use Survey by Landau Associates (2000). Logs for some of these wells apparently contain erroneous well locations, so the wells were located by Landau Associates using street addresses. The water supply source for the "DOS Water System" (Group B public water system) is in this vicinity and was not differentiated or mapped separately from the other wells. The DOS Water System has 10 connections.

Table 4 presents the number of water right certificates, permits, applications, and claims for the quarter-quarter sections within 0.25-mile of the proposed trenches, based on a query of Ecology's WRATS database. Water rights listed for those sections, but that do not have quarter-quarter location information, are not included on the table.

INFILTRATION TESTING AND DATA ANALYSIS

This section summarizes infiltration testing results and interpretation. Four infiltration tests were conducted to provide empirical data on infiltration effects, and allow estimation of horizontal and vertical hydraulic conductivities of the shallow MTA by calibration of groundwater models. The test locations are shown on **Figure 2. Appendices A and B** present test results for infiltration tests I-1 and I-2 that were interpreted in detail. Infiltration tests I-3 and I-4 were conducted in areas of lower hydraulic conductivity and shallow water tables; long-term infiltration rates were shown to be severely limited by groundwater mounding at those locations. Therefore, interpretation of I-3 and I-4 data was limited and infiltration facilities were not proposed for those areas.

Infiltration Test Procedures and Results

Infiltration tests were conducted by discharging potable water into 12-ft-by-12-ft square boxes imbedded in shallow soils. At sites I-1 and I-2, twelve piezometers were installed in six boreholes located 2, 10, and 50 feet from two edges of the infiltration box. In general, one shallow and one deep piezometer were installed in each borehole. The shallow set of piezometers consisted of 2-foot-long well screens installed just below the water table. The deeper set of piezometers consisted of 2-foot long well screens typically 10 to 15 feet below the shallower set. Seals between the sand packs were composed of bentonite. **Table 5** presents piezometer construction data compiled by others.

Water was discharged to the infiltration test basins at a nearly constant rate, and water levels were monitored in the piezometers both by hand and by pressure transducers. Water discharge continued until groundwater levels in the piezometers no longer changed substantially. **Table 6** summarizes the site I-1 infiltration test duration, steady-state discharge rate, and steady-state mound heights interpreted from the data presented in **Appendix A**. **Table 7** presents the same information for infiltration test site I-2, based on the data presented in **Appendix B**.

The infiltration site I-3 test was conducted at a location characterized by boring B-6 (**Figure 2**). Surface soils were generally finer (less permeable) compared to sites I-1 and I-2 and pre-test depth-to-groundwater was only 2.5 feet. Test duration was 136 hours, but infiltration ceased after about 32 hours when the water level in the basin reached ground surface (a ponding depth of about 18 inches within the basin). The average infiltration rate over the 136 hours was about 0.5 gpm. Infiltration was limited by the aquifer's ability to transmit water away from the infiltration basin with the mound height limited to 2.5 feet.

The infiltration site I-4 test was conducted at a location characterized by boring B-7 (**Figure 2**). Surface soils were generally finer compared to I-1 and I-2, and pre-test depth to groundwater was only 5.5 feet. Test duration was 168 hours but virtual cessation of infiltration occurred after about 24 hours with water levels in the basin at ground surface (a ponding depth of about 24 inches within the basin). The average infiltration rate over the 168 hours was about 0.4 gpm. Infiltration was limited by the aquifer's ability to move water away from the infiltration basin, in part due to the limited mounding height of 5.5 feet.

MODELING OF TREATED EFFLUENT AND STORMWATER INFILTRATION

Local and regional MODFLOW models were developed to simulate infiltration of treated effluent and stormwater. Local models were used to assess maximum mound heights under the infiltration facilities. The regional model was used to calculate interference between the numerous effluent and stormwater infiltration features and particle travel times to streams. The mound height components predicted by the local and regional models were added to average background groundwater elevations to calculate mounded groundwater elevations. To assess infiltration feasibility, the mounded groundwater elevations were compared to elevations of critical site features, as provided by others. This section reviews the groundwater models. A later section summarizes the infiltration feasibility assessment.

Local Groundwater Models

Two groups of local models were developed. First, local models of the I-1 and I-2 infiltration tests were developed to calculate vertical and horizontal hydraulic conductivity, based on calibration to the test data. The names of these local models are **Tulalip Infiltration 1 (site I-1)** and **Tulalip Infiltration 2 (site I-2). Table 8** indicates the horizontal and vertical hydraulic conductivities calculated from calibration of the models to the mounding measured during the tests (**Tables 6 and 7**).

A second set of local models was used to calculate local maximum groundwater mounding heights at proposed stormwater basins and trenches. Local models were used because maximum mound heights are sensitive to model cell size and the regional model cells were too large for desirable accuracy. This second group of local models consists of:

- **Five-foot wide trench models** (Infiltration Trenches **1A-1C**): Steady state numerical models to simulate two-dimensional groundwater mounding.
- **Stormwater Basins (Casino North** and **Casino South**): Steady state numerical models designed to simulate three-dimensional groundwater mounding for two large rectangular stormwater basins.

All local groundwater models consisted of four layers. Layer elevations were assigned to enhance output detail for specific elevations as follows: layer 1 (the top layer) between 80 and 100 ft, layer 2 between 70 and 80 ft., layer 3 between 60 and 70 feet, and layer 4 was between 0 and 60 feet. The initial heads for the models began at 83 feet; providing an initial saturated thickness of three feet for the top layer and fully saturated thicknesses for all other layers. The aquifer was simulated as unconfined, and all recharge was assumed to reach the water table. The other model boundary conditions and parameters are summarized in **Table 8.**

Regional Groundwater Model

A regional steady-state groundwater model was developed using MODFLOW to simulate the interference among infiltration facilities, with consideration for the hydraulic influence of local streams and wetlands. The model domain is divided into 3-dimensional cells in 3 layers, 150 rows, and 115 columns (**Figure 6**). Horizontal hydraulic conductivity was set to 48 ft/day in layers 1 and 2, and to 32 ft/day in layer 3, through the calibration process. These values are consistent with aquifer testing data discussed earlier, but are lower than the average. Vertical hydraulic conductivity was reduced relative to the horizontal values to represent interbedded silt. The target saturated thickness of the upper layer under calibration conditions was limited to 5-to-10 feet to minimize errors from vertical averaging of the mound heights. The combined thickness of the three layers varied from 153 to 220 feet, which was intended to simulate the full thickness of the Marysville Trough Aquifer, although actual thickness is not well characterized. In plan view, the smallest cells are 20 by 50 feet horizontally and were used near the infiltration facilities. The largest cells were 200 by 200 feet.

Sturgeon, Coho, and Quil Ceda Creeks were simulated within appropriate model layers using the river package. River cell heads were defined using survey data provided by Parametrix and Landau Associates, where available, and by interpolation from USGS topographic quadrangles, where survey data were not available.

Recharge to the ambient-condition model was distributed to all cells in the top row at a rate of 0.006 ft/day (26.28 inches per year).

The groundwater elevation point values (wells) that were used to develop the contours of **Figure 2**, also were used as calibration targets to guide model adjustments and to define river cell heads and bottom elevations. Model calibration was achieved when simulated groundwater elevations matched observed values to the extent possible, using reasonable parameter values **(Figure 7)**. **Figure 8** compares contours of observed versus modeled groundwater elevations and indicates that although good agreement was achieved at the calibration targets (wells), agreement is not as good at some more-remote locations.

Groundwater elevations at numerous critical "observation points," shown in **Figure 9,** were simulated by the calibrated (ambient condition) regional model. The observation points consist of infiltration facilities, building locations, and monitoring wells.

The following changes were then made in the model to simulate future conditions:

- Precipitation recharge was eliminated in the area of the project to simulate impervious surfaces associated with the development.
- Stormwater infiltration was simulated by the addition of injection wells at stormwater facilities.
- Treated effluent infiltration was simulated by the addition of injection wells at effluent facilities.

Table 9 presents the assumed future stormwater and effluent infiltration rates as specified by Parametrix for this feasibility assessment. Since the models are steady-state, the stormwater and effluent infiltration rates are single values and approximate the average of the time-variant reality. The stormwater rates are intended to represent wet season average conditions, as calculated by Parametrix when assuming that two-thirds of the 36-inches of annual rainfall occurs in the wettest six months. The effluent infiltration rates were based on a total wastewater volume of 250,000 gallons per day, which represents average wastewater flows.

Figure 9 is a map of the predicted groundwater table configuration for the modeled future condition. Predicted groundwater flowlines to Quil Ceda Creek are shown for several locations along the proposed infiltration trenches. The model predicts that groundwater travel times between the trenches and Quil Ceda Creek range from 245 to

1207 days. The travel times generally correlate to the flowline length shown on **Figure 9**. The average travel times from the northern and southern infiltration trenches to the creek are 379 days and 1059 days, respectively. The travel times on the south end may be underestimated by the model as a result of the model gradient being steeper than predicted by contouring of field data in that area (no direct measurements of groundwater elevation are available in that area).

Groundwater elevations at the observation points used for the ambient condition were also simulated for the future condition. The ambient condition elevations were subtracted from the future condition elevations to compute groundwater mound heights that are predicted to result from the stormwater and effluent infiltration, as modeled. **Figure 10** is a map of the height of the groundwater mounds as predicted by the regional model for the modeled future condition. As discussed in the following section, this regional model prediction probably underestimates mound height in the immediate vicinity of infiltration facilities.

INFILTRATION FEASIBILITY ASSESSMENT

Four groundwater factors were considered in estimating total groundwater elevations at the critical locations:

- maximum mound heights as simulated by the local models
- interference mounding from surrounding facilities as calculated by the regional model
- groundwater elevations measured in (or interpolated from) the December 2001 through January 2002 field measurements, and
- estimated ambient maximum groundwater elevation

The derivation of these factors is discussed in preceding sections of this report. This section describes how the factors were combined into the feasibility assessment.

The value of each factor at each observation point was compiled on a spreadsheet (**Table 10**). The first set of columns on **Table 10** present the names of the observation points, critical elevations for those points, and rationale for assigning the critical elevations. The critical elevations generally represent elevations at which shallow groundwater may become a problem, as specified by Parametrix. The second set of columns presents natural groundwater elevation components. The third set of columns present stormwater and effluent-infiltration mounding components. Lastly, the fourth set of columns summarizes the total elevations and compares the model-predicted future groundwater elevations to the critical elevations. Shading on the table highlights locations where predicted groundwater elevations are above critical elevations.

The analysis suggests that future groundwater elevations will likely remain below critical elevations at 39 of the 45 observation points. Conversely, future groundwater elevations are likely to exceed critical elevations at 6 observation points. The conditions at these 6 locations are discussed below.

Critical groundwater elevations are expected to be exceeded at the Casino stormwater infiltration basins and a nearby footing drain (observation points CSN01, CSN02, and CSN03). **Table 10** indicates that the predominant mounding factor at these locations is stormwater infiltration at the Casino basins (3 to 4 feet of mounding at the local basin plus interference mounding from the nearby basin). Infiltration of treated wastewater is predicted to contribute about 0.75 feet of groundwater mounding at the CSN01 and CSN02 stormwater basins under the conditions modeled. Estimated worst-case *ambient* groundwater elevations are predicted to exceed the critical elevation at CSN03. In response to these predictions, monitoring wells have been installed at the basins to improve site specific groundwater information. The additional information will be considered in detailed design of the stormwater features, which may accommodate occasional high groundwater levels.

Critical groundwater elevations are also expected to be exceeded at the Home Depot stormwater infiltration basin (HD1). Examination of **Table 10** indicates that natural groundwater elevations exceed the critical elevation and that the combined stormwater and treated effluent infiltration will result in about 1.6 feet of groundwater mounding. Water (probably groundwater) was observed in this stormwater basin after a period of no rainfall in the winter of 2001-2002.

Although the critical groundwater elevation is predicted to be exceeded at observation point 99str1, the stormwater feature associated with the observation point at that location has been moved to avoid shallow groundwater. Therefore the critical elevation at that location is no longer applicable, and the indication of exceedence is moot.

REFERENCES

AGRA Earth & Environmental, Inc., 1996. Subsurface Exploration and Geotechnical Engineering Study, 27th Avenue NE Sewer, 88th Street NE to 196th Street NE, Marysville, Washington. February.

Economic and Engineering Services, Inc., 1991, Snohomish County Groundwater Characterization Study. Prepared for Snohomish County, July 1991.

Landau Associates, 1999. Phase II Data Report, Boeing Tulalip Test Site, Marysville, Washington. Prepared for the Boeing Company. October 6.

Landau Associates, 2000, Beneficial Water Use Survey, Tulalip Test Site, Marysville WA. Memorandum Prepared for the Boeing Company. August 3 2000.

Shannon and Wilson, 1994. Draft Geotechnical Report, SR-5/88th Street N.E. Interchange, Marysville, Washington. March.

Thomas, B.E., J.M. Wilkinson, and S.S. Embrey of U.S. Geological Survey, 1997. The Ground-Water System and Ground-Water Quality in Western Snohomish County, Washington. Water-Resources Investigations Report 96-4312

Wert and Associates, 1995. New Casino For Tulalip Tribes: Soil Study for On-Site Treatment and Reuse of Domestic Wastewater. March.

Table 1. Summary of Marysville Trough Stratigraphy in Project Vicinity

Name of Layer	Material Description	Thickness (ft)	Elevation of Top (ft, msl)
Vashon Recessional Deposits (Qvr)	Fine to Medium Sand with occasional Silt in matrix, and in layers	• 50 to 150 regionally • < ~70 (USGS, 1997) • >80 (Landau, 1998, 1999)	50 to 70 (land surface)
Vashon Till (Qvt)	Cemented Silt, Sand, and Gravel	Unknown, possibly absent (5 to 100 regionally)	Possibly absent
Vashon Advance Deposits (Qva)	Sand and Gravel fining downward to Fine Sand	Unknown (up to 350 regionally)	
Transition Beds (Qtb)	Clay, Silt and Fine Sand		0 (USGS, 1997) < -100 (Landau, 1999)

Table 2. Summary of Pumping Test Analysis Results

		Cooper-Jacob Analysis Parameters					Theim Analysis Parameters						
Piezometer	Aquifer Response and Data Period	discharge (gpm)	slope (feet per cycle)	thickness (ft)	T (ft2/day)	K (ft/day)	discharge (gpm)	R (ft)	r (ft)	H (ft)	h in feet (50% well efficiency)	K (ft/day)	Comments
P-1	Trans. Pumping	1.2	0.07	12	652	55							
P-1	Trans. Recovery	1.2	0.06	12	770	65							back-flow influence early in recovery'
P-2	Trans. Pumping	1.1	0.27	9	144	16							
P-2	Trans. Recovery	1.1	0.17	9	228	26							back-flow influence early in recovery'
P-3	Trans. Pumping	2.7	0.30	15	318	21							
P-3	Recovery	2.7	NA	NA	NA	NA							data not usable
P-4	Trans. Pumping	2.6	0.08	13	1101	84							
P-4	Trans. Recovery	2.6	0.16	13	562	43							back-flow influence early in recovery'
P-5	Steady Pumping						3.8	25	0.25	13	12.4	25	
P-5	Trans. Pumping	3.8	0.08	15	1788	116							
P-6	Trans. Pumping	5.5	0.85	18	228	13							good data
P-6	Trans. Recovery	5.5	0.80	18	243	14							good data
P-7	Trans. Pumping	5.5	0.23	17	863	50							good data, latest data only
P-7	Trans. Recovery	5.5	0.30	17	647	37							good data
P-8	Trans. Pumping	5.5	0.18	18	1099	60							good data, latest data only
P-8	Trans. Recovery	5.5	0.23	18	856	47							good data
P-9	Steady Pumping						5.5	25	0.25	16	15	36	10-15 minutes
P-9	Trans. Recovery	5.5	0.08	16	2532	156							curved response
				median		47							
				mean		54							
				max		156							
				min		13							

Table 3. Summary of Groundwater Level Measurements

Location (Well Name)	Top of Monument Elevation (feet - NGVD29)	Depth to groundwater (feet)	Groundwater Elevation (feet)	Depth to groundwater (feet)	Groundwater Elevation (feet)	Depth to groundwater (feet)	Groundwater Elevation (feet)	Depth to groundwater (feet)	Groundwater Elevation (feet)	Depth to Groundwater (feet)	Groundwater Elevation (feet)	Depth to Groundwater (feet)	Groundwater Elevation (feet)	Depth to Groundwater (feet)	Groundwater Elevation (feet)
P-1	54.95	9.18	45.77	8.62	46.33	7.2	47.75	Destroyed	N/A						
P-2	51.66	12.20	39.46	11.71	39.95	11.7	39.96	Destroyed	N/A	Destroyed	N/A	Destroyed	N/A	Destroyed	N/A
P-3	63.30	6.16	57.14	5.46	57.84	5.2	58.10	4.6	58.70	9.66	53.64	9.37	53.93	9.68	53.62
P-4	52.05	8.44	43.61	7.44	44.61	6.4	45.65	5.5	46.60	4.75	47.30	4.51	47.54	4.63	47.42
P-5	47.94	5.98	41.96	5.06	42.88	4.6	43.34	4.1	43.89	6.16	41.78	5.91	42.03	6.31	41.63
P-6	53.37	N/A	N/A	2.60	50.77	2.8	50.57	2.6	50.77	4.57	48.80	4.35	49.02	4.58	48.79
P-7	49.57	3.94	45.63	3.19	46.38	3.0	46.57	2.6	46.97	2.75	46.82	2.65	46.92	3.05	46.52
P-8	54.18	2.47	51.71	2.14	52.04	2.5	51.68	2.3	51.88	2.88	51.30	2.73	51.45	2.98	51.20
P-9	51.80	6.56	45.24	5.32	46.48	4.5	47.30	3.8	48.05	2.57	49.23	2.45	49.35	2.66	49.14
B-1	61.31	18.85	42.46			16.9	44.41	16.2	45.16	4.38	56.93	4.13	57.18	4.59	56.72
B-2	57.39	17.17	40.22			14.9	42.49	14.3	43.14	15.72	41.67	15.09	42.30	14.51	42.88
B-3	51.96	15.70	36.26			13.7	38.26	13.0	38.96	14.40	37.56	13.86	38.10	14.27	37.69
B-4	54.47	5.60	48.87			4.0	50.47	3.4	51.07	12.93	41.54	12.48	41.99	12.77	41.70
B-5	58.42	3.80	54.62			3.1	55.32	2.6	55.87	3.46	54.96	3.51	54.91	4.43	53.99
B-6	52.77	4.48	48.29			3.5	49.27	3.1	49.72	2.77	50.00	3.09	49.68	3.57	49.20
B-7	49.86	5.72	44.14			4.1	45.76	3.6	46.26	3.40	46.46	3.22	46.64	3.57	46.29
I-1 E1	62.23	19.23	43.00							3.70	58.53	3.82	58.41	3.93	58.30
I-1 E2	62.43	19.52	42.91			17.4	45.03	16.7	45.73						
I-1 E3	61.80	19.15	42.65					16.5	45.35	16.26	45.54	15.64	46.16	15.02	46.78
I-1 S1	61.99	18.95	43.04							Disturbed	N/A	Disturbed		Disturbed	
I-1 S2	62.48	19.45	43.03												
I-1 S3	62.40	19.42	42.98					16.6	45.80						
I-2 E1	51.74	13.40	38.34							16.20	35.54	15.54	36.20	14.90	36.84
I-2 E2	51.81	13.55	38.26			11.6	40.21	10.7	41.16						
I-2 E3	51.97	13.95	38.02					11.0	40.97	10.79	41.18	10.41	41.56	10.79	41.18
I-2 S1	51.68	13.25	38.43												
I-2 S2	51.66	13.25	38.41												
I-2 S3	51.72	13.35	38.37					10.6	41.17						
I-3 W1	51.33	3.10	48.23												
I-3 W2	51.37	3.02	48.35					1.7	49.67						
I-3 S1	51.25	3.03	48.22												
I-3 S2	51.66	3.45	48.21												
I-3 S3	51.16	2.86	48.30					1.7	49.46						
TGW-017	68.26					12.4		11.2	57.04	11.72	56.54	11.23	57.03	11.26	57.00
TGW-024	67.39					5.9		5.2	62.19	6.14	61.25	6.15	61.24	6.56	60.83
TGW-036								5.1							
TGW-039	75.71					12.9		12.2	63.51	13.62	62.09	13.41	62.30	13.66	62.05
TGW-058								8.7							
TGW-059								16.1							
TGW-061								21.7							

Notes: Groundwater depths were measured from the top of monument except TGW-xxx wells were measured from top of casing.
Measurements by AMEC

Table 4. Water Rights Confirmed within 0.25-Miles of Infiltration Trenches

Section	Quarter Sections	Number of Groundwater:			
		Certificates	Permits	Applications	Claims
16	nwnw,swnw,nwsw,swsw	1	3	0	0
17	nene,sene,nese,sese	0	0	0	0
20	nene	0	0	0	0
21	nwnw	2	1	0	0
9	swsw	0	0	0	0
8	sese	0	0	0	0

Other certificates, permits, applications, and claims exist in the subject sections without specified quarter sections.

Table 5. Summary of Well Construction Data

Well ID	Date Completed	Well type	Depth to groundwater [1,2] (feet)	Depth to top of upper seal (feet)	Depth to bottom of upper seal (feet)	Depth to top of upper filter pack (feet)	Depth to bottom of upper filter pack (feet)	Screened interval-upper (feet)	Depth to top of lower seal (feet)	Depth to bottom of lower seal (feet)	Depth to top of lower filter pack (feet)	Depth to bottom of lower filter pack (feet)	Screened interval-lower (feet)
B-1	11/6/01	2" piezo	18.85	1	7	7	20.5	9.5-19.5	N/A	N/A	N/A	N/A	N/A
B-2	11/5/01	2" piezo	17.17	2	7	7	24	10-20	N/A	N/A	N/A	N/A	N/A
B-3	11/7/01	2" piezo	15.70[3]	1	7	7	20	10-20	N/A	N/A	N/A	N/A	N/A
B-4	11/8/01	2" piezo	5.60	1	6.5	6.5	20	10-20	N/A	N/A	N/A	N/A	N/A
B-5	11/12/01	2" piezo	3.80	1	5.5	7	20	10-20	N/A	N/A	N/A	N/A	N/A
B-6	11/13/01	2" piezo	4.48	1	6.5	7	20.5	10-20	N/A	N/A	N/A	N/A	N/A
B-7	11/9/01	2" piezo	5.72	1	6.5	6.5	20	10-20	N/A	N/A	N/A	N/A	N/A
P-1	11/7/01	2" piezo	9.18	1.5	7.5	7.5	20.5	9.5-19.5	N/A	N/A	N/A	N/A	N/A
P-2	11/7/01	2" piezo	12.20	1.5	7	7	20.5	9.5-19.5	N/A	N/A	N/A	N/A	N/A
P-3	11/7/01	2" piezo	6.16	1	7.5	7.5	20.5	10-20	N/A	N/A	N/A	N/A	N/A
P-4	11/7/01	2" piezo	8.44	1.5	8	8	20.5	10-20	N/A	N/A	N/A	N/A	N/A
P-5	11/7/01	2" piezo	5.98	1.5	7	7	20.5	9.5-19.5	N/A	N/A	N/A	N/A	N/A
P-6	11/8/01	2" piezo	N/A[4]	1.5	7	7	20.5	10-20	N/A	N/A	N/A	N/A	N/A
P-7	11/8/01	2" piezo	3.94	1	7	7	20.5	10-20	N/A	N/A	N/A	N/A	N/A
P-8	11/8/01	2" piezo	2.47	1	7	7	20.5	9.5-19.5	N/A	N/A	N/A	N/A	N/A
P-9	11/14/01	2" piezo	6.56	1.5	7.5	7.5	21.5	10-20	N/A	N/A	N/A	N/A	N/A
I-1/S-1	11/13/01	2, 1" piezos	18.95	1.5	10	10	25.5	13-15	25.5	30	30	35.5	33-35
I-1/S-2	11/13/01	2, 1" piezos	19.45	1.5	15	15	22	18-20	22	30	30	35.5	33-35
I-1/S-3	11/13/01	2, 1" piezos	19.42	1.5	17	17	26	20-22	26	30	30	35.5	33-35
I-1/E-1	11/14/01	2, 1" piezos	19.23	1.5	17	17	25	20-22	25	30	30	35.5	33-35
I-1/E-2	11/14/01	2, 1" piezos	19.52	1.5	17.9	17.9	24.8	20-22	24.8	30	30	35.5	33-35
I-1/E-3	11/14/01	2, 1" piezos	19.15	1.5	17	17	25.2	20-22	25.2	30	30	35.5	33-35
I-2/S-1	11/12/01	2, 1" piezos	13.25	1.5	10	10	25.5	13-15	25.5	30	30	35.5	33-35
I-2/S-2	11/12/01	2, 1" piezos	13.25	1	10	10	20	13-15	20	30	30	35.5	33-35
I-2/S-3	11/13/01	2, 1" piezos	13.35	1	10	10	20	13-15	20	30	30	35.5	33-35
I-2/E-1	11/12/01	2, 1" piezos	13.40	1.5	10.2	10.2	20	13-15	20	30	30	35.5	33-35
I-2/E-2	11/12/01	2, 1" piezos	13.55	1.5	9.8	9.8	20	13-15	20	30	30	35.5	33-35
I-2/E-3	11/13/01	2, 1" piezos	13.95	1.5	10	10	20	13-15	20	30	30	35.5	33-35
I-3/S-1	11/8/01	2, 1" piezos	3.03	1.5	10	10	18	13-15	18	28	28	35.5	33-35
I-3/S-2	11/9/01	2, 1" piezos	3.45	1.5	10	10	17	13-15	17	30	30	35.5	33-35
I-3/S-3	11/8/01	2, 1" piezos	2.86	1.5	10	10	18	13-15	18	26	26	35.5	33-35
I-3/W-1	11/9/01	2, 1" piezos	3.10	1.5	10.5	10.5	22	13-15	22	30	30	35.5	33-35
I-3/W-2	11/9/01	2, 1" piezos	3.02	1.5	10.2	10.2	19	13-15	19	30	30	35.5	33-35

Notes: 1. All depth measurements referenced to the top of the steel monument (approximate ground surface).
2. Depth to groundwater measurements obtained 11/20/01
3. Measurement taken on 11/12/01
4. No measurement obtained
5. All wells installed by AMEC. This table compiled by AMEC.

Table 6. Summary of Infiltration Test I-1 Data

Discharge Duration:	98.48 hours
Average Discharge Rate:	29.8 gpm
Average Pre-Test Depth to Groundwater:	19 feet below ground surface

Piezometer	Approximate Radial Distance and Depth to Center of Screen (ft)	Steady-State Mound Height in feet (nearest 0.05-ft)	Average Steady-State Mound Height for pairs with common radial distance and depth (ft)
I1/E1S	2, 14	2.0	2.0
I1/S1S		Dry	
I1/E1D	2, 34	1.5	1.5
I1/S1D		1.5	
I1/E2S	10, 14	1.4	1.5
I1/S2S		1.6	
I1/E2D	10, 34	1.2	1.25
I1/S2D		1.3	
I1/E3S	50, 14	0.6	0.7
I1/S3S		0.8	
I1/E3D	50, 34	0.6	0.75
I1/S3D		0.9	

Table 7. Summary of Infiltration Test I-2 Data

Discharge Duration:	149.13 hours
Average Discharge Rate:	7.8 gpm
Average Pre-Test Depth to Groundwater:	13 feet below ground surface

Piezometer	Approximate Radial Distance and Depth to Center of Screen (ft)	Steady-State Mound Height in feet (nearest 0.05-ft)	Average Steady-State Mound Height for pairs with common radial distance and depth (ft)*
I2/E1S	2, 14	0.9	0.85
I2/S1S		0.8	
I2/E1D	2, 34	0.2	0.2
I2/S1D		0.15?	
I2/E2S	10, 14	0.4	0.4
I2/S2S		0.4	
I2/E2D	10, 34	0.2	0.2
I2/S2D		0.2	
I2/E3S	50, 14	0.2	0.2
I2/S3S		0.15	
I2/E3D	50, 34	0.2	0.2
I2/S3D		0.15	

** also considers confidence in data, values rounded up to nearest 0.05-feet*

Table 8. **Summary of Local Model Properties**

Model	Geometry				K Values (Kx/Kz) (ft/day)		Boundary Condition
	No. Cells	Model Domain (ft)	Recharge Area (ft)	Center Cell Dimension (ft)	Layer 1-3	Layer 4	
Infiltration Test Models							
I-1	125 x 125	500 x 500	12 x 12	4 x 4	80/55	50/1	GHB
I-2	125 x 125	500 x 500	12 x 12	4 x 4	75/4.5	50/1	GHB
5' Trench Models							
1A	1 x 300	5 x 2485	5 x 5	0.5 x 5	80/55	50/1	GHB
1C	1 x 300	5 x 2485	5 x 5	0.5 x 5	75/4.5	50/1	GHB
1D	1 x 300	5 x 2485	5 x 5	0.5 x 5	75/4.5	50/1	GHB
Stormwater Basin Models							
Casino North	256 x 299	2290 x 2280	140 x 80	5 x 5	80/55	50/1	GHB
Casino South	257 x 300	2290 x 2280	300 x 80	5 x 5	80/55	50/1	GHB

Table 9. Modeled Effluent and Stormwater Infiltration Rates

Modeled Treated Effluent Infiltration System Discharges

Basin		Length (ft)	Width (ft)	Average Flow (gpd)	Average Inf. Rate (ft/day)	Average Inf. Rate (ft3/day)	Area (acres)
1	A1	1200	5	79,079	1.762	10,572.00	0.14
	A2	800	5	39,539	1.322	5,286.00	0.09
	C1	625	5	30,890	1.322	4,129.69	0.07
	C2	625	5	41,187	1.762	5,506.25	0.07
	D	900	5	59,309	1.762	7,929.00	0.10
Total		3250		250,004		33,423	0.48

Modeled Wet Season Stormwater Infiltration Rates

Location	Estimated Annual Avg. Wet Season SW Flow (cfs)	Basin Length (feet)	Basin Width (feet)	Flow Rate (cfs)	Infiltration Rate (ft/s)	Infiltration Rate (ft/day)
SE Retail #1	0.008	30	6	0.008	4.42E-05	3.82
SE Retail #2	0.012	50	6	0.012	3.98E-05	3.44
Walmart #1	0.028	350	6	0.028	1.35E-05	1.16
Walmart #2	0.049	600	6	0.049	1.35E-05	1.16
Walmart #3	0.022	410	4	0.022	1.35E-05	1.16
Home Depot	0.070	300	100	0.070	2.32E-06	0.20
Casino - North	0.238	140	80	0.238	2.12E-05	1.83
Casino - South	0.211	300	80	0.211	8.81E-06	0.76
Chelsea (Planned)	0.241	1000	20	0.241	1.21E-05	1.04
99th Street	0.014	150	20	0.014	4.81E-06	0.42
Quilceda Parkway	0.008	100	10	0.008	7.96E-06	0.69

Table 10. Summary of Modeled Groundwater Mound Heights and Elevations

Key parameters (flows are sums of actual model input and may differ slightly from specifications):

Natural condition assumption:	March 99 (worst case)
SS scenario:	model version 22
SS stormwater infiltration rate:	wet season flows
SS effluent infiltrtation rate at 1A1 (note 9)	82,399 gpd
SS effluent infiltrtation rate at 1A2	37,144 gpd
SS effluent infiltrtation rate at 1C1	30,451 gpd
SS effluent infiltrtation rate at 1C2	41,663 gpd
SS effluent infiltrtation rate at 1D	59,309 gpd
Sum of effluent flows =	250,966 gpd

Field and Model Results

Observation Point Data Provided by Parametrix							Natural Water Table Elevation Components			SW and E Mounding Components			Summaries		
Location Number	Location Description	Symbol on Map	Ground Elevation or Other (ft NGVD29)	Adjustment (ft)	Reason for Adjustment or Reason for Inclusion if No Adjustment	Maximum allowed (critical) groundwater elevation (ft)	Estimated Maximum Water Table Elevation Winter 01-02 (ft)	Estimated Worst Case Additional Water Table Rise (ft)	Estimated Worst Case natural Water Table Elevation (ft)	Local Mound Calculated With Local Model (ft)	Regional Mound From Combined SW and E Infiltration (ft)	Reduction Where Local Mound was Double Counted (ft)	Estimated Total Mound Height Above Natural Elevation (ft)	Estimated Total Groundwater Elevation (ft)	Estimated Elevation Over Critical Elevation (ft)
			Note 1				Note 2, 5	Note 3	Note 4, 5			Note 7		Note 5	Note 5,6
1	Boeing Administration Area 1	BAA1	64.2	0.0	Assess effects on TCE plume	64.2	45	2.6	48		0.4		0.4	48	-16.2
2	North Area 1	N1	64.1	0.0	Assess effects on TCE plume	64.1	45	2.6	48		0.7		0.7	48	-15.8
3	Chelsea 1 - Center of Site	CH1	62.7	-4.0	Estimated Elevation Footing Drains	58.7	47	2.6	50		1.1		1.1	51	-8.0
5	Casino Stormwater Inf. Basin - North	CSNO1	47.0	0.0	SW Basin Base	47.0	43	2.6	46	4.0	4.8	2.8	6.0	52	4.6
6	Casino Stormwater Inf. Basin - South	CSNO2	46.0	0.0	SW Basin Base	46.0	42	2.6	45	2.9	4.1	2.0	5.0	50	3.6
7	Casino Bldg Footing Drain NW @ N31	CSN03	51.4	0.0	Foot Drain Invert	51.4	51	2.6	54		0.9		0.9	54	3.1
8	Casino Bldg Footing Drain NE @ N22	CSNO4	52.0	0.0	Foot Drain Invert	52.0	48	2.6	51		1.2		1.2	52	-0.2
9	Casino Bldg Footing Drain SE @ S16	CSNO5	50.0	0.0	Foot Drain Invert	50.0	46	1	47		1.3		1.3	48	-1.7
10	Home Depot Stormwater Inf. Basin 1	HD1	34.0	0.0	SW Basin Base	34.0	35	1	36		1.6		1.6	38	3.6
11	Walmart Stormwater Inf. Basin 1	WM1	37.0	0.0	SW Basin Base	37.0	24	1	25		0.9		0.9	26	-11.1
12	Walmart Stormwater Inf. Basin 2-A	WM2	39.0	0.0	SW Basin Base	39.0	23	2.6	26		0.9		0.9	27	-12.5
13	Walmart Stormwater Inf. Basin 2-B	WM3	39.0	0.0	SW Basin Base	39.0	27	2.6	30		1.2		1.2	31	-8.2
14	Walmart Stormwater Inf. Basin 3	WM4	40.0	0.0	SW Basin Base	40.0	29	1	30		1.1		1.1	31	-8.9
15	SE Retail Stormwater Inf. Basin 1	SER1	33.0	0.0	SW Basin Base	33.0	22	2.6	25		0.8		0.8	25	-7.6
16	SE Retail Stormwater Inf. Basin 2	SER2	33.0	0.0	SW Basin Base	33.0	22	1	23		0.6		0.6	24	-9.4
17	99th Street Stormwater Inf. Basin 1 (note 8)	99Str1	48.2	0.0	SW Basin Base	48.2	48	1	49		1.0		1.0	50	1.8
18	Quilceda Parkway Stormwater Inf. Basin 1	QCPW1	43.0	0.0	SW Basin Base	43.0	40	1	41		1.0		1.0	42	-1.0
19	WW Effluent Inf. Trench 1A 1	EIT1A1	62.0	-4.0	Trench (4 ft Deep)	58.0	45	2.6	48	1.5	0.9	1.1	1.4	49	-9.0
20	WW Effluent Inf. Trench 1A 2	EIT1A2	60.5	-4.0	Trench (4 ft Deep)	56.5	44	2.6	47	1.5	1.5	1.1	1.9	49	-8.0
21	WW Effluent Inf. Trench 1A 3	EIT1A3	60.0	-4.0	Trench (4 ft Deep)	56.0	43	2.6	46	1.5	1.7	1.1	2.1	48	-8.3
22	WW Effluent Inf. Trench 1A 4	EIT1A4	59.2	-4.0	Trench (4 ft Deep)	55.2	43	2.6	46	1.1	1.9	0.8	2.2	48	-7.4
23	WW Effluent Inf. Trench 1A 5	EIT1A5	57.0	-4.0	Trench (4 ft Deep)	53.0	42	2.6	45	1.1	2.7	0.8	3.1	48	-5.3
24	WW Effluent Inf. Trench 1B 1	EIT1B1	55.0	-4.0	Trench (4 ft Deep)	51.0	42	2.6	45	0.0	2.5	0.0	2.5	47	-3.9
25	WW Effluent Inf. Trench 1B 2	EIT1B2	52.8	-4.0	Trench (4 ft Deep)	48.8	42	2.6	45	0.0	2.4	0.0	2.4	47	-1.8
26	WW Effluent Inf. Trench 1C 1	EIT1C1	55.2	-4.0	Trench (4 ft Deep)	51.2	40	2.6	43	1.3	2.5	0.9	2.9	46	-5.7
27	WW Effluent Inf. Trench 1C 2	EIT1C2	53.0	-4.0	Trench (4 ft Deep)	49.0	37	2.6	40	1.3	2.1	0.9	2.5	42	-6.9
28	WW Effluent Inf. Trench 1C 3	EIT1C3	51.5	-4.0	Trench (4 ft Deep)	47.5	34	2.6	37	1.7	2.0	1.2	2.5	39	-8.4
29	WW Effluent Inf. Trench 1D 1	EIT1D1	50.2	-4.0	Trench (4 ft Deep)	46.2	31	2.6	34	1.7	1.9	1.2	2.4	36	-10.2
30	WW Effluent Inf. Trench 1D 2	EIT1D2	47.2	-4.0	Trench (4 ft Deep)	43.2	25	2.6	28	1.7	1.4	1.2	1.9	29	-13.7
31	WW Effluent Inf. Trench 2A 1	EIT2A1	51.7	-4.0	Trench (4 ft Deep)	47.7	41	1	42	0.0	1.6	0.0	1.6	44	-4.1
32	WW Effluent Inf. Trench 2A 2	EIT2A2	51.7	-4.0	Trench (4 ft Deep)	47.7	38	1	39	0.0	1.3	0.0	1.3	40	-7.4
33	WW Effluent Inf. Trench 2A 3	EIT2A3	48.3	-4.0	Trench (4 ft Deep)	44.3	35	1	36	0.0	1.2	0.0	1.2	37	-7.1
34	GW Monitoring Well B1 (East Chelsea)	GWB1	61.3	0.0	Future Comparison: Model to Reality	61.3	44.37	2.6	47		1.5		1.5	48	-12.8
35	GW Monitoring Well B2 (East Casino)	GWB2	57.4	0.0	Future Comparison: Model to Reality	57.4	42.45	2.6	45		2.2		2.2	47	-10.1
36	GW Monitoring Well B3 (East, 3rd Retail Site)	GWB3	51.9	0.0	Future Comparison: Model to Reality	51.9	38.22	2.6	41		1.9		1.9	43	-9.2
37	GW Monitoring Well B4 (NW Coho Creek)	GWB4	54.4	0.0	Future Comparison: Model to Reality	54.4	50.64	1	52		0.7		0.7	52	-2.1
38	GW Monitoring Well B5 (West Casino)	GWB5	58.4	0.0	Future Comparison: Model to Reality	58.4	56.09	1	57		0.5		0.5	58	-0.8
39	GW Monitoring Well P1 (99th & Quilceda Way)	GWP1	54.9	0.0	Future Comparison: Model to Reality	54.9	47.72	1	49		1.1		1.1	50	-5.0
40	GW Monitoring Well P2 (West, 3rd Retail Site)	GWP2	51.6	0.0	Future Comparison: Model to Reality	51.6	40.62	1	42		1.2		1.2	43	-8.8
41	GW Monitoring Well P3 (West of Chelsea)	GWP3	63.3	0.0	Future Comparison: Model to Reality	63.3	58.06	2.6	61		1.0		1.0	62	-1.6
42	GW Monitoring Well I1 E1 S (East Chelsea)	GWI1E1S	62.2	0.0	Future Comparison: Model to Reality	62.2	44.99	2.6	48		1.3		1.3	49	-13.3
43	GW Monitoring Well I2 E1 S (Center, 3rd Retail)	GWI2E1S	51.7	0.0	Future Comparison: Model to Reality	51.7	40.17	1	41		1.4		1.4	43	-9.2
44	Interstate 5 Ditch (East of Chelsea)	I51	51.0	0.0	Prevent Discharge to I-5 Ditch (Estimated)	51.0	44	2.6	47		1.3		1.3	48	-3.1
45	Interstate 5 Ditch (East of Casino)	I52	48.0	0.0	Prevent Discharge to I-5 Ditch (Estimated)	48.0	42	2.6	45		2.3		2.3	47	-1.1
46	Interstate 5 Ditch (East of 3rd Retail)	I53	42.0	0.0	Prevent Discharge to I-5 Ditch (Estimated)	42.0	36	2.6	39		2.1		2.1	41	-1.3

notes
1 All elevations are to Tribal datum as surveyed by PMX (NGVD29)
2 Natural elevations are from contoured water table elevations.
3 Interpreted as 2.6 ft or 1.0 ft. 2.6 feet is the water table elevation difference between winter 01-02, and historical maximum in March 1999, in Landau well TGW-017. 1.0 is a reduced value where depth to water is low (would flood) or where streams are nearby.
4 Estimated historical maximum natural water table elevation (March 1999).
5 Values in shaded cells exceed maximum elevation criteria established by Parametrix.
6 A negative value indicates that the predicted elevation remains below the maximum allowed elevation.
7 Estimated as 70% of the local model maximum mound height
8 This stormwater facility was moved west to avoid shallow groundwater. The observation point was not moved.
9 Locations of the portions of the trenches labeled 1A1, 1A2, 1C1, etc., are shown on Figure 6.

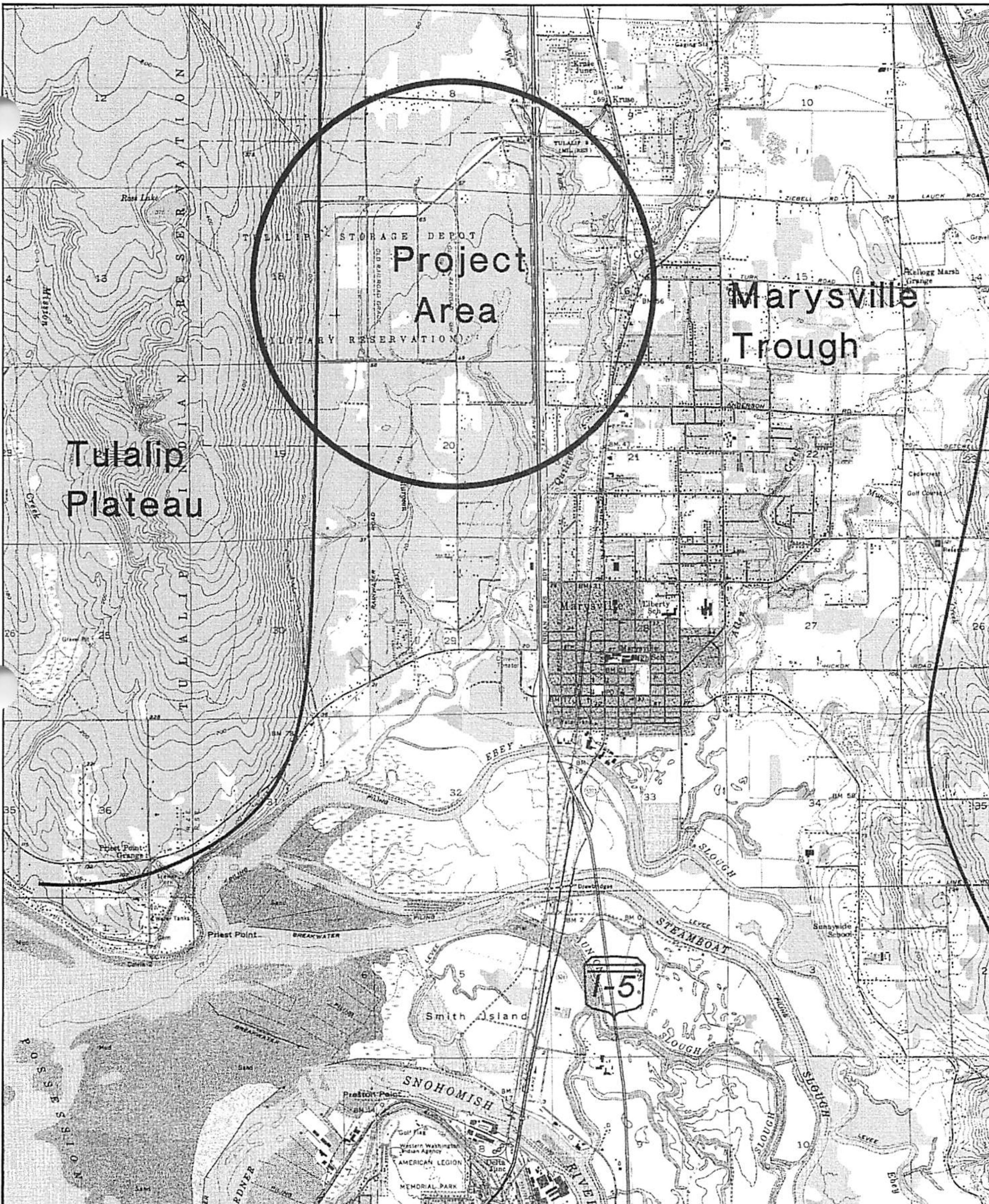
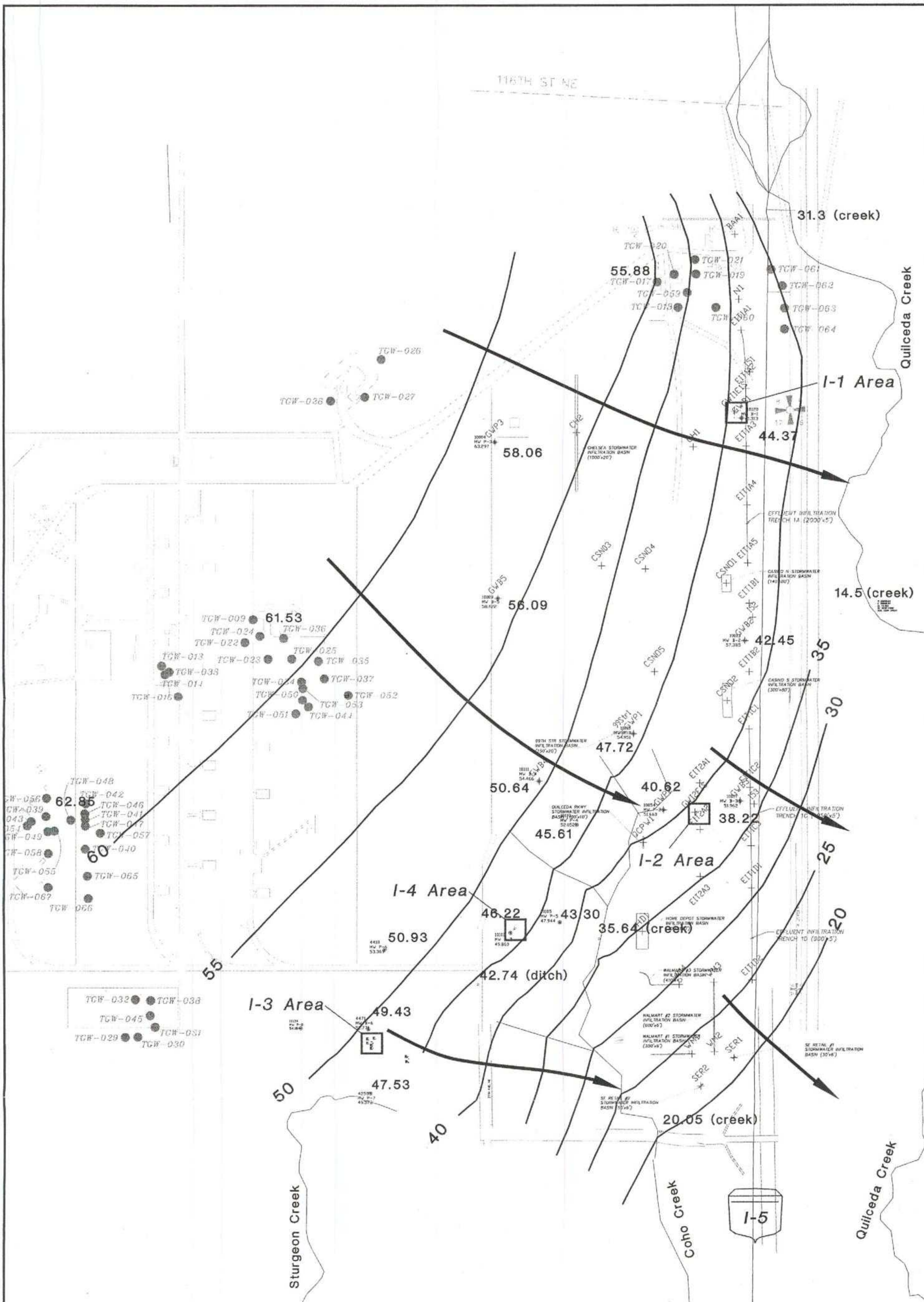


FIGURE 1
Regional Geologic Setting

Tulalip Construction Project

Pacific Groundwater Group

JE0108. PGG_Figure_1.dwg



LEGEND

- Landau Monitoring Well
- Tribe Monitoring Well
- 47.53 Maximum Observed Water Elevation in winter 01-02 (NGVD 29)
- 40 — Groundwater Elevation Contour
- SER2 + Observation Point and Label
- → Groundwater Flow Direction

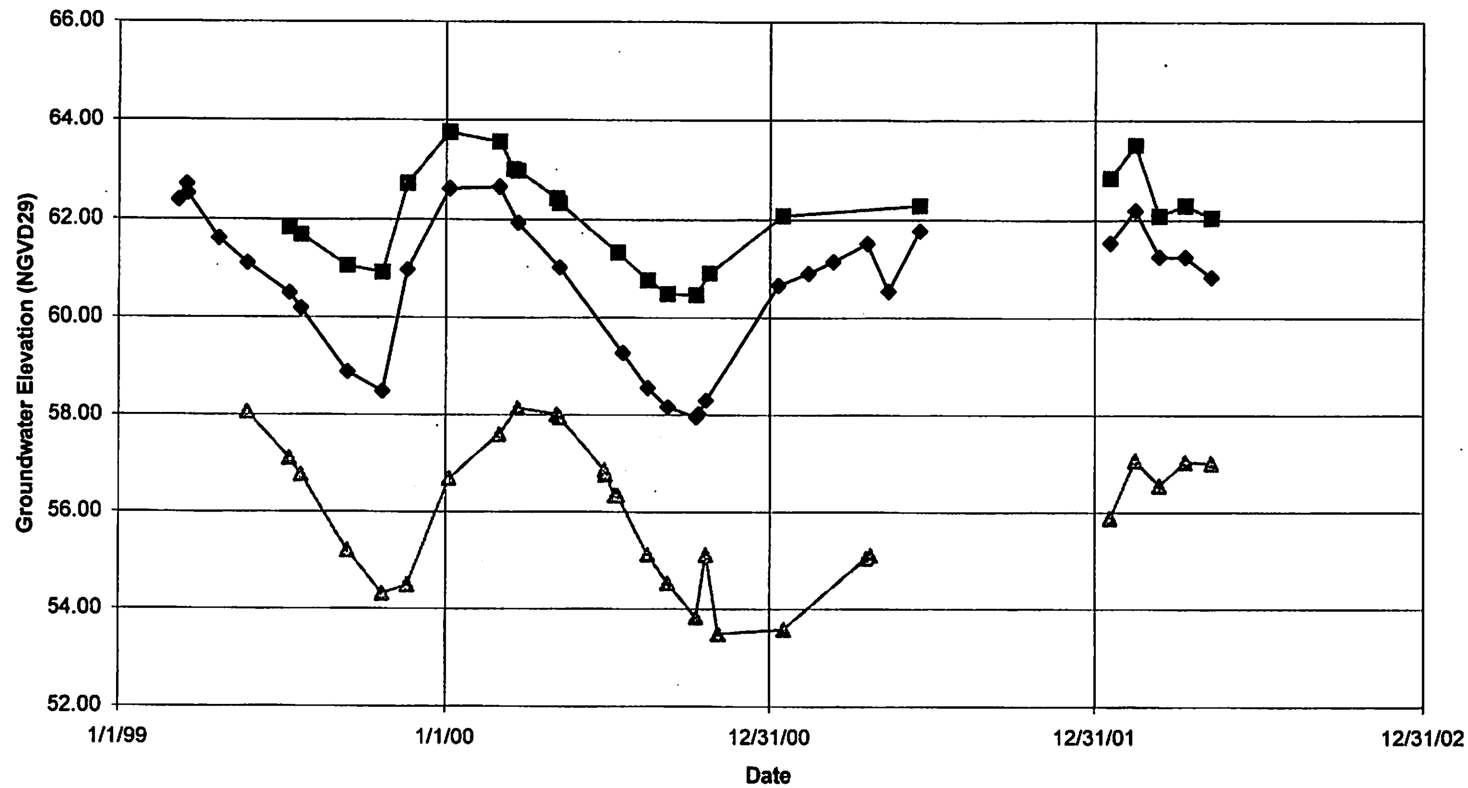
0 400 800
Scale in Feet

FIGURE 2
Groundwater Flow Direction in the Marysville Trough Aquifer

Tulalip Construction Project

Pacific Groundwater Group

JE0108, PGG_figure-2.dwg



TGW-024
TGW-039
TGW-017

FIGURE 3
Selected Groundwater Hydrographs

Pacific Groundwater Group

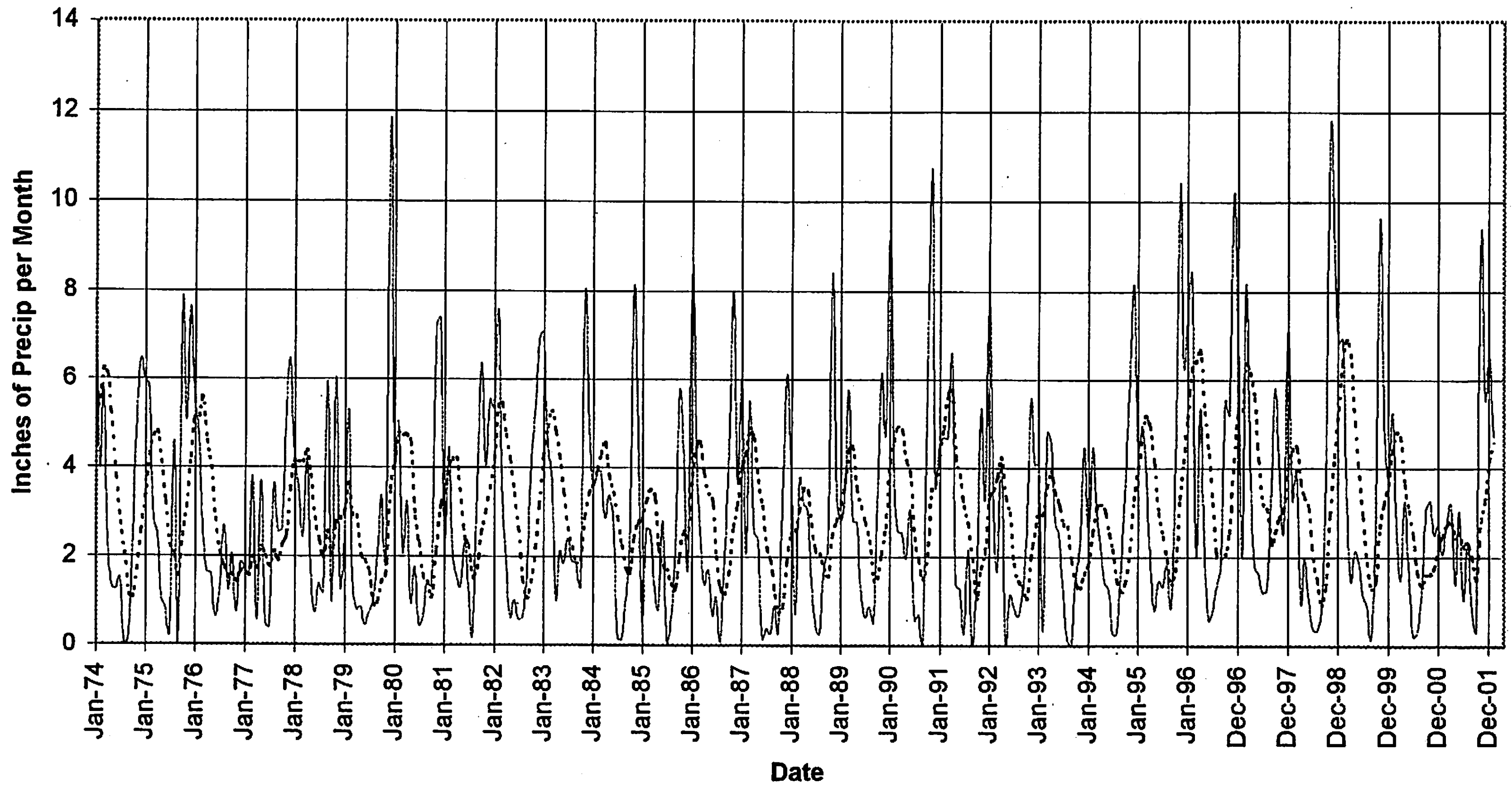
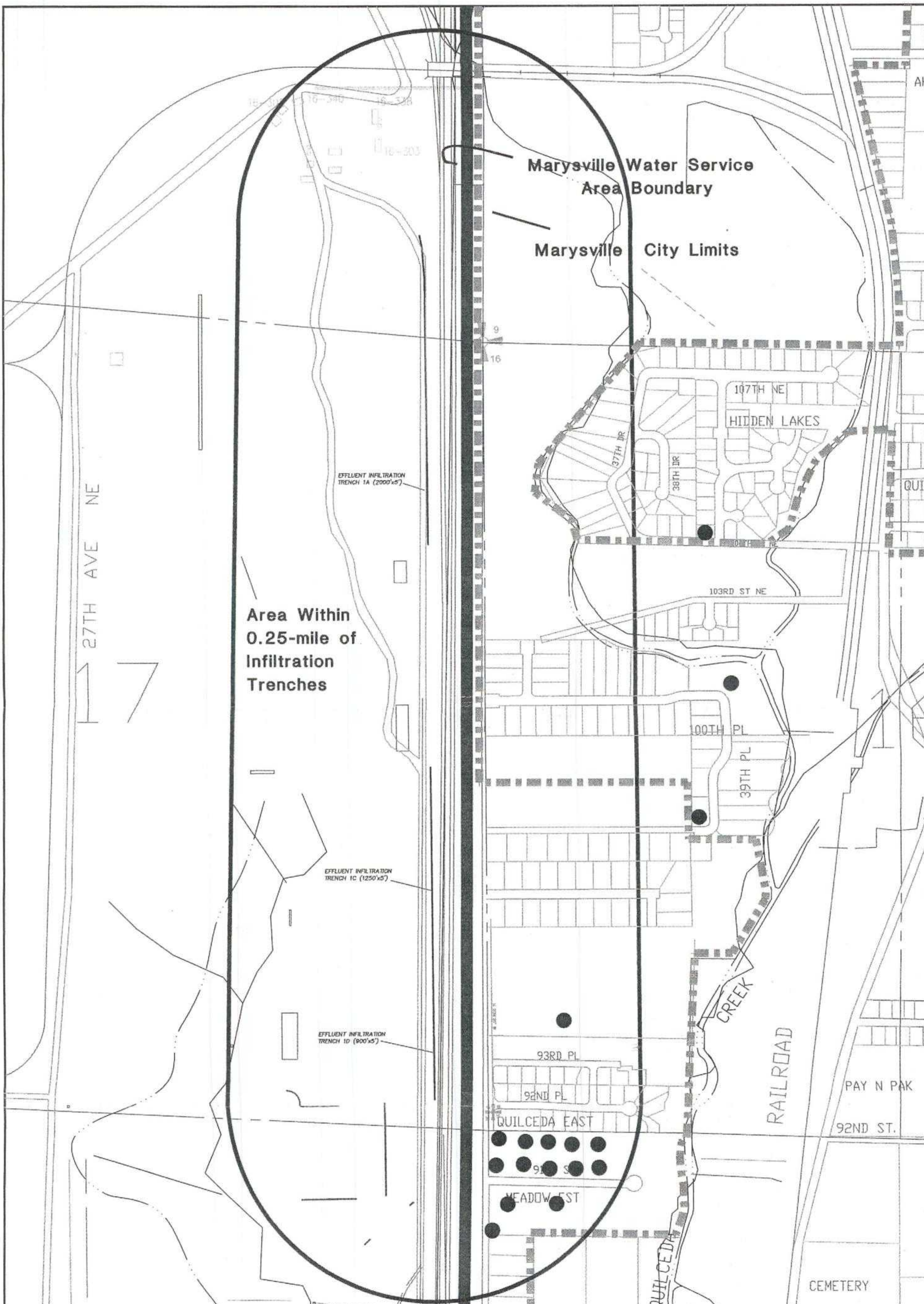


FIGURE 4
Monthly Precipitation at SeaTac

Pacific Groundwater Group



LEGEND

● Reported Water Well

FIGURE 5
Water Supply Wells within 0.25-mile of Project

Tulalip Construction Project

Pacific Groundwater Group

JE0108, water well map.dwg

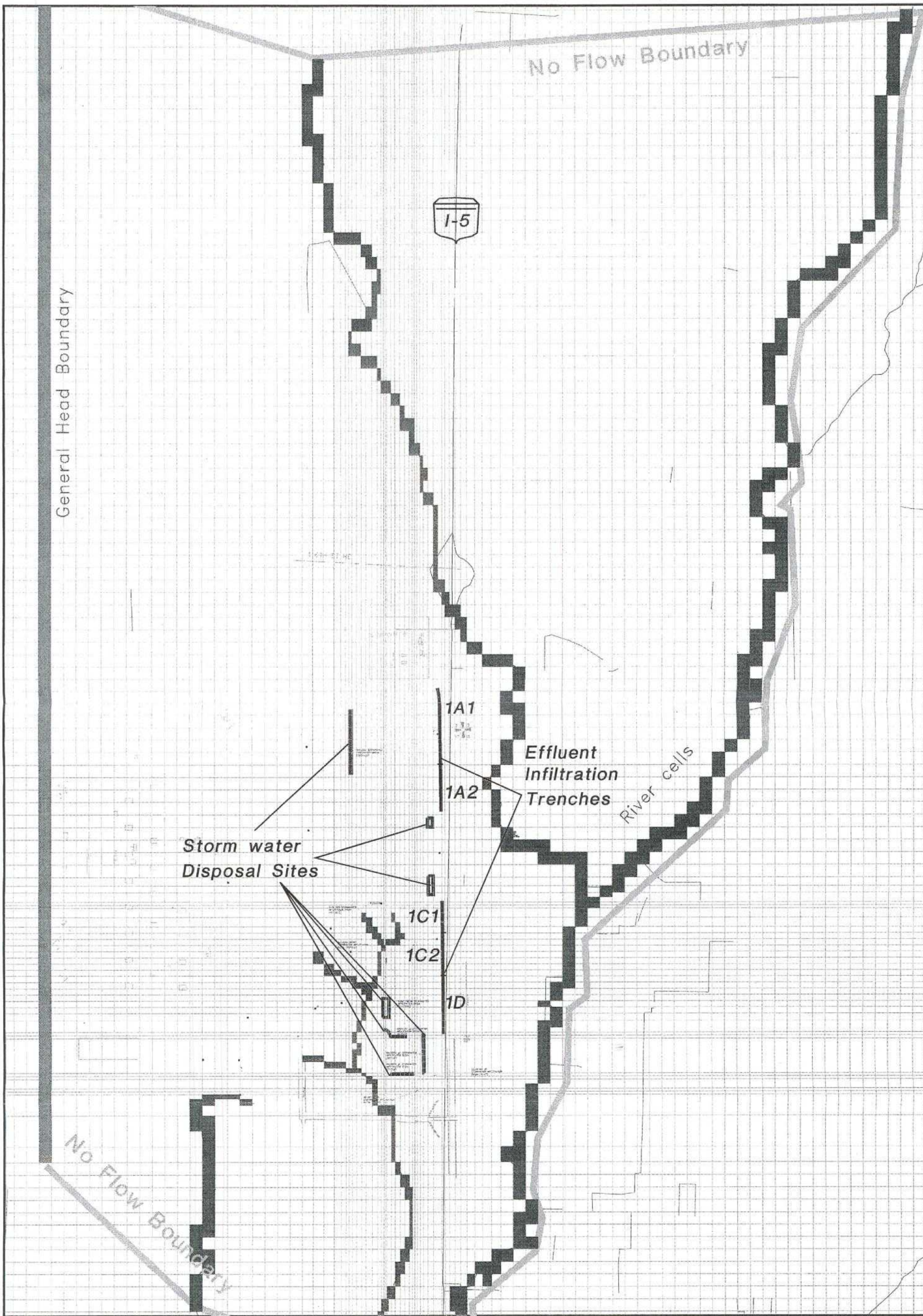


FIGURE 6
Grid of Regional MODFLOW Model

Tulalip Construction Project

Pacific Groundwater Group

JE0105, P00_figure_6 regional grid.dwg

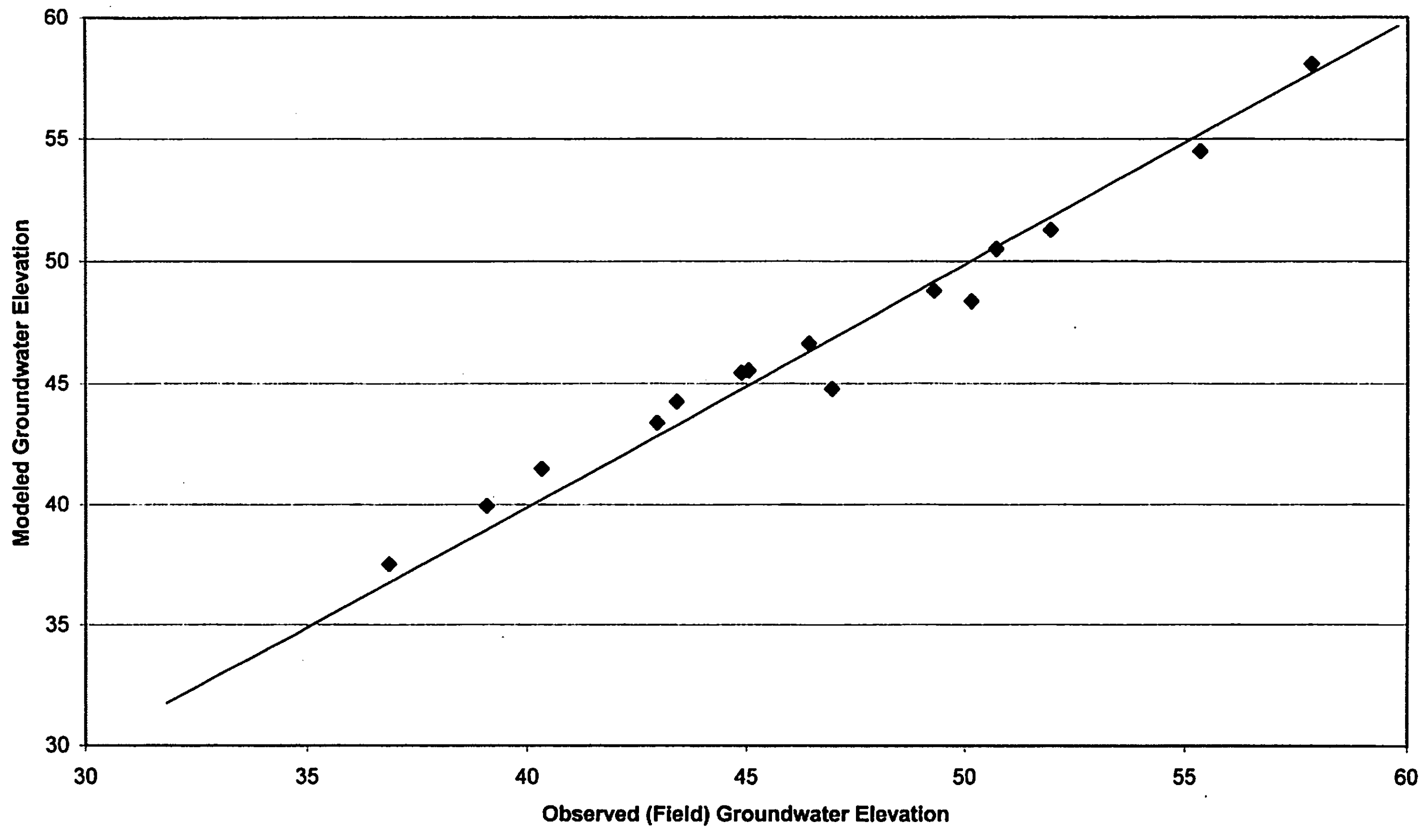
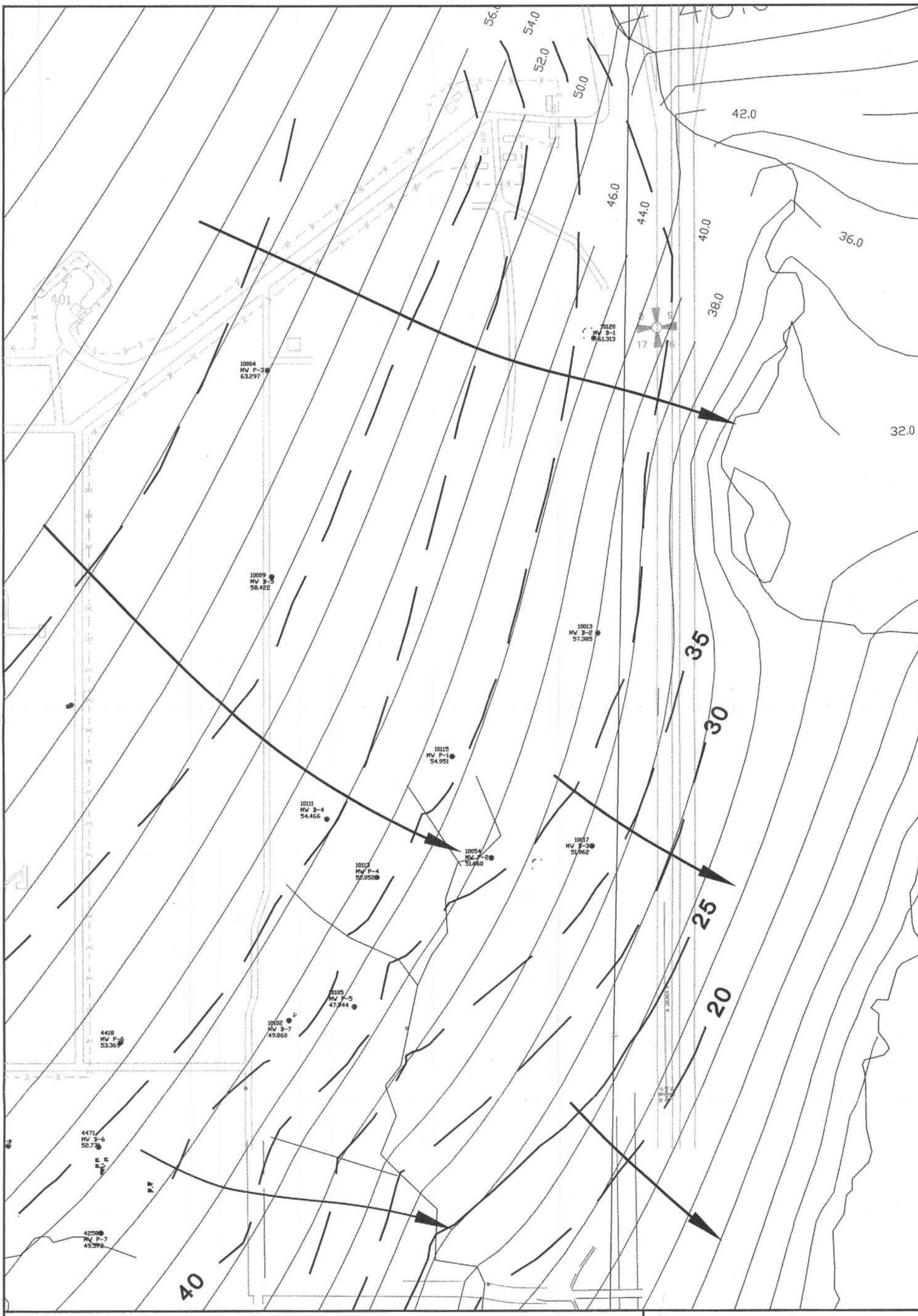


FIGURE 7
Graph of Observed Versus Modeled Groundwater Elevations

Pacific Groundwater Group



LEGEND

●	Tribe Monitoring Well	
47.53	Maximum Observed Water Elevation observed winter 01-02	
40 ▬	Groundwater Elevation Contour (from measured data)	
52.0 —	Groundwater Elevation Contour (model calibration run 11)	
→	Groundwater Flow Direction	

N

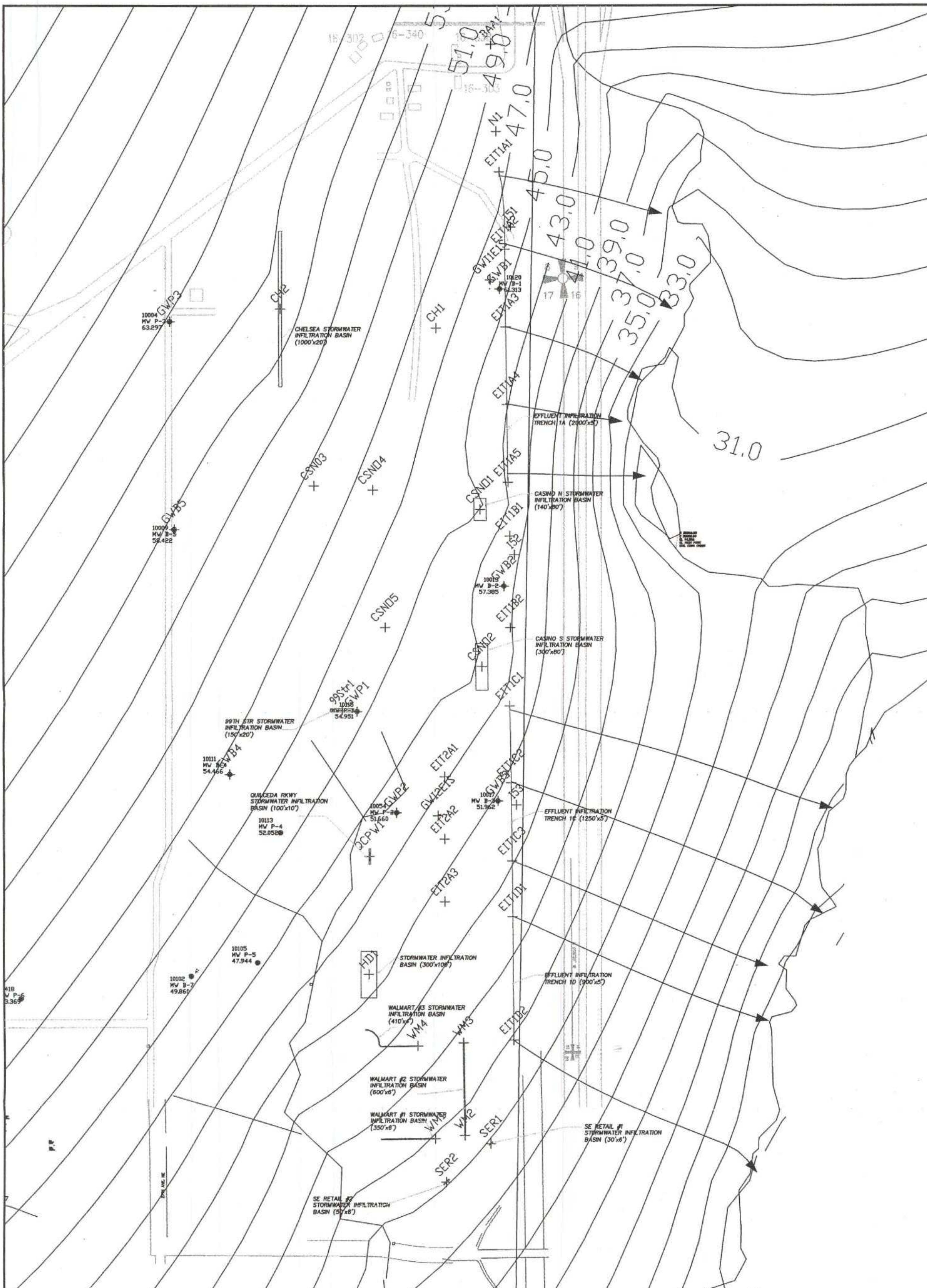
0 300 600
Scale in Feet

FIGURE 8
Map of Observed Versus Modeled Groundwater Elevations

Tulalip Construction Project

Pacific Groundwater Group

JE0108, PGG_figure_8.dwg



LEGEND

Groundwater Flowline from Infiltration Trench

40.0 — Groundwater Elevation Contour (ft, NGVD29)

SER2 + Observation Point and Label

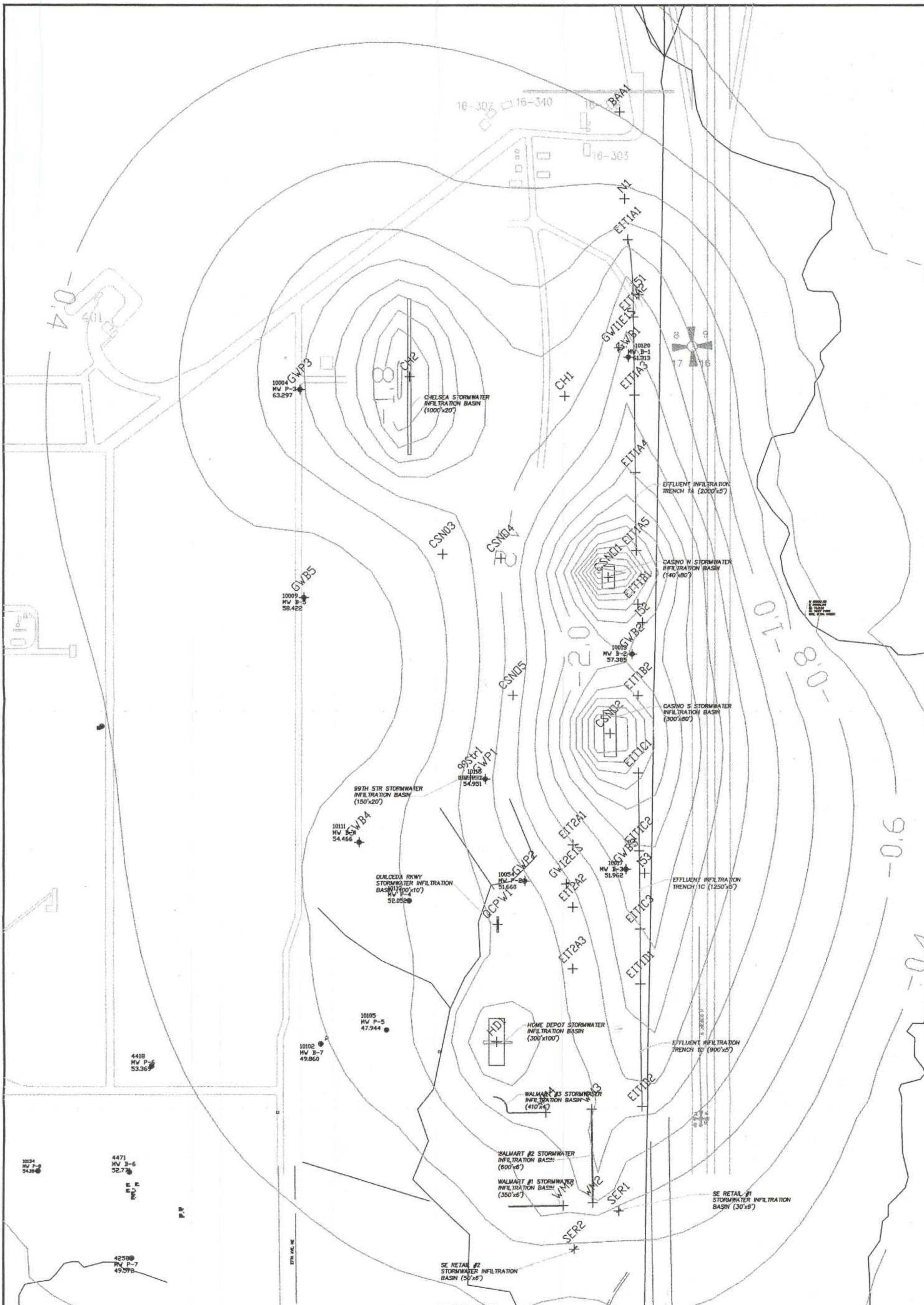
0 300 600
Scale In Feet

FIGURE 9
Map of Modeled Future Groundwater Elevation and Flowlines

Tulalip Construction Project

Pacific Groundwater Group

JE0106, PGG_Figure_9.dwg



LEGEND

- Tribe Monitoring Well
- SER2 + Observation Point and Label
- -2.0 — Groundwater Mound Height Contour (ft)

0 300 600
Scale in Feet

FIGURE 10
Map of Modeled Future Mound Height

Tulalip Construction Project

Pacific Groundwater Group

JE0108, PGG_Figure_10.dwg

Appendix A

I-1 Local Hydrogeologic Conditions

Boring B-1, advanced by AMEC on November 6 2001, is located near infiltration test I-1, and serves as the best exploration of stratigraphy at that location. B-1 encountered the following materials below ground surface (bgs) to total depth of 51.5 feet:

Depth Range	B-1 Material Description
0 to 10.5 feet bgs	Fine-to-medium SAND, grading downward to medium-to-coarse SAND.
10.5 to 15.5 feet bgs	2-inch lens of sandy SILT, underlain by interbedded fine-to-medium SAND and fine sandy SILT
15.5 to at least 51.5 feet bgs	Fine-to-medium SAND, with trace-to-some silt, and one thin silt bed at 45 feet

Twelve piezometers were installed in six boreholes surrounding I-1. Piezometers were installed in boreholes 2, 10, and 50 feet from the east edge of the 12x12-foot infiltration basin. Another set of piezometers was installed in boreholes advanced 2, 10, and 50 feet from the south edge of the basin. Soils were logged by observing soils coming off the auger flights, but no soil samples were collected ahead of the auger. The construction of the piezometers is summarized in **Table 5** of the main text. In general, one shallow and one deep piezometer were installed in each borehole. The shallow set of piezometers consists of 2-foot-long well screens from 20 to 22 feet depth (with two exceptions – see **Table 5**), which are surrounded by coarse sand packs from 17 to 25 feet depth (with three exceptions). The deeper set of piezometers consist of 2-foot long well screens from 33 to 35 feet depth, surrounded by sand packs from 30 to 35 feet depth. Seals between the sand packs are composed of bentonite.

Pre-test water levels in the piezometers indicate that depth to the water table was approximately 19 feet bgs and that little vertical flow was occurring (heads in shallow and deep piezometers were similar).

Water was discharged to the I-1 infiltration test basin for 98.48 hours between November 30 and December 4, 2001. Discharge rate was a nearly constant 29.8 gallons per minute (gpm) into the 144 square-foot basin.

Figures A1 through A7 show water levels in nearby piezometers as the test progressed. **Figures A1 and A2** most clearly indicate the mounding effects, water level recovery after mounding, and the regional water level increases that occurred over the test duration because of substantial precipitation. The trends on **Figures A1 and A2** were used to estimate the rate of regional water level increase over the testing period. Solid lines were drawn on all I-1 plots to represent the regional (background) water level rise, starting at the intersection of the data and the on-set of mounding. Dotted lines with the same slope as the regional trend were drawn through a graphically-estimated average steady-state water level after mounding stabilized. The distance between the two lines is the average steady-state water level increase caused by infiltration. Those mound heights are summarized in **Table 6** of the main text.

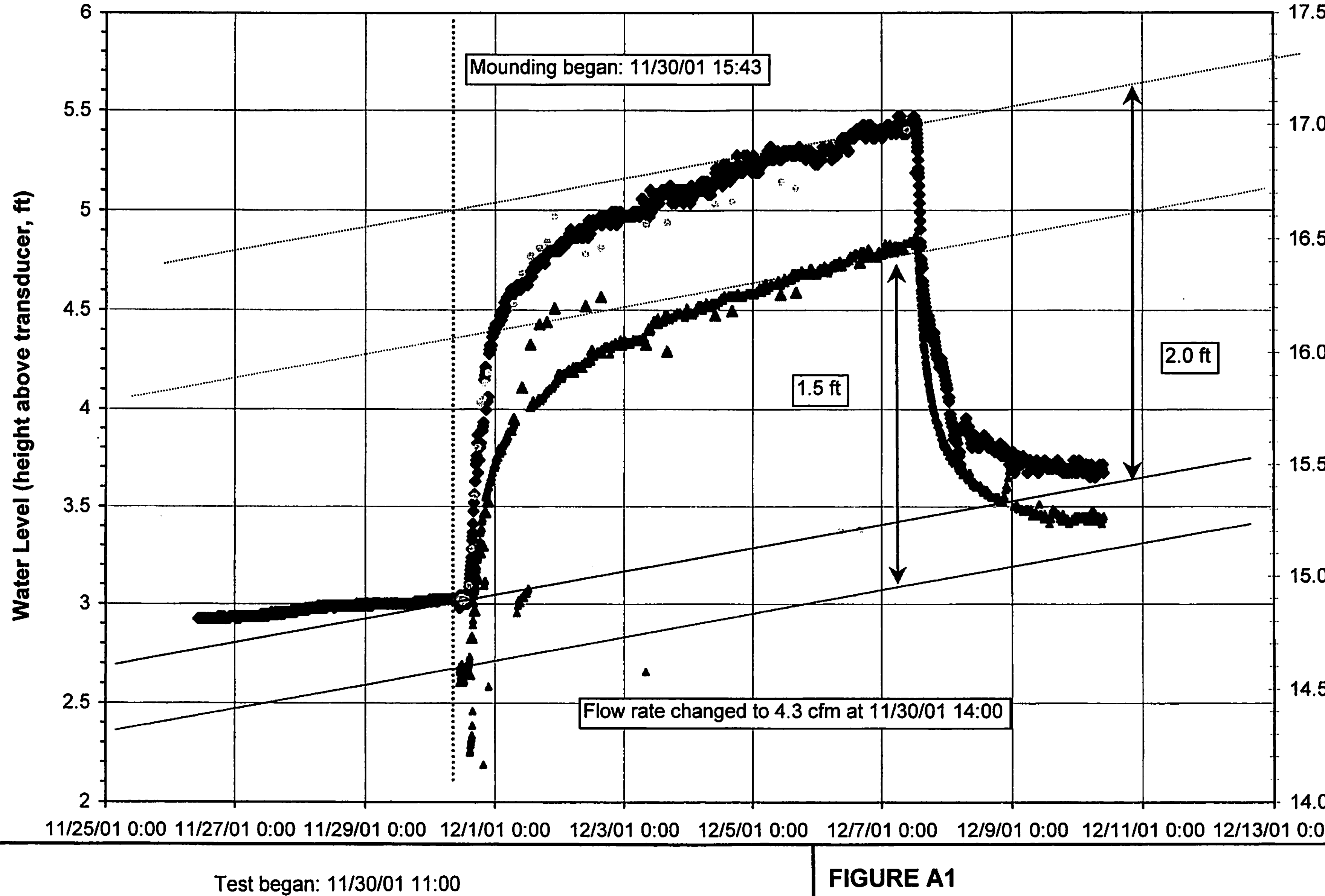


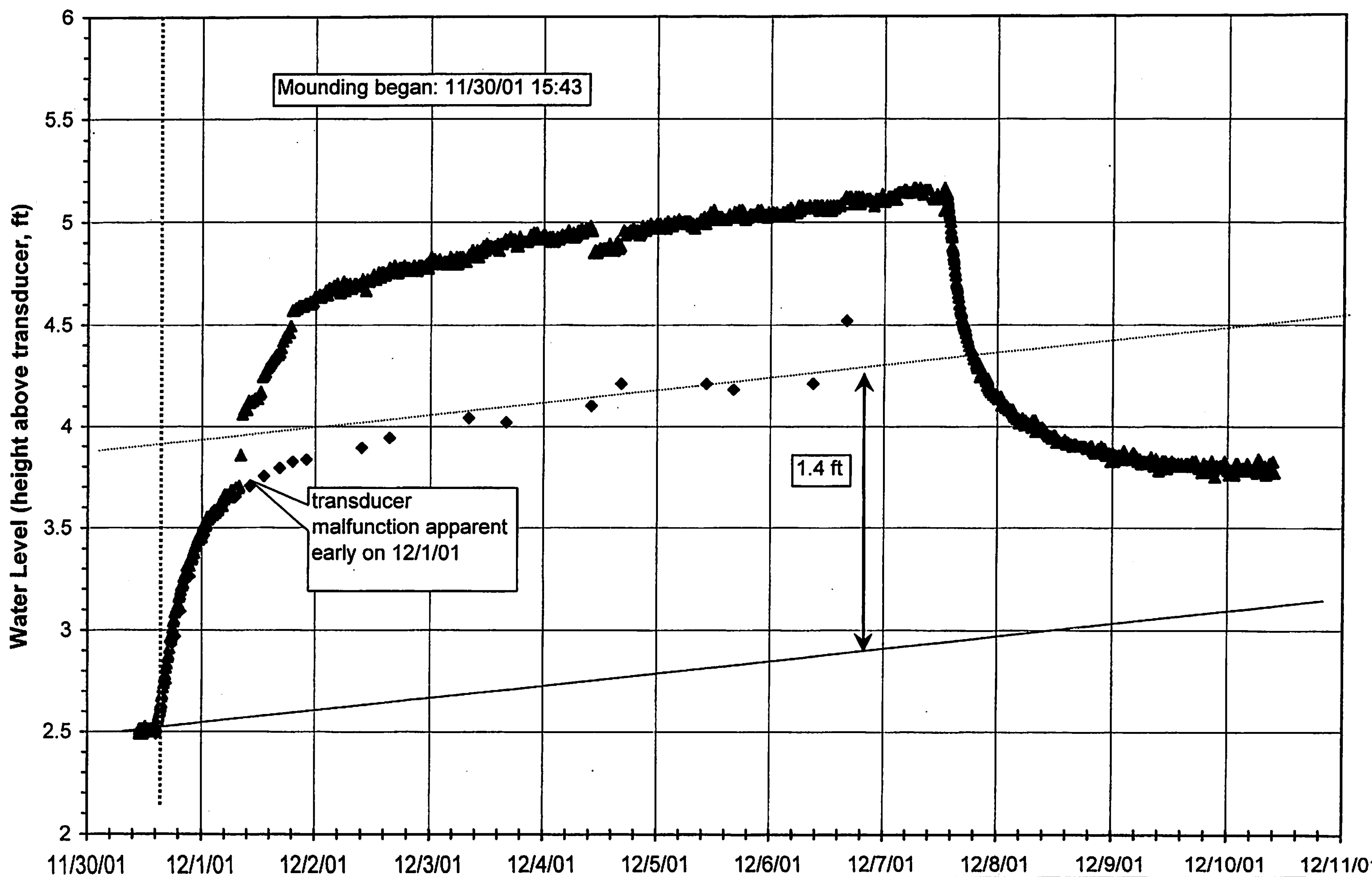
FIGURE A1
Water Levels in Piezometer I1/E1

Tulip Infiltration Tests
I1 Test

Pacific Groundwater Group

5/10/02

I1 Arithmetic Plots.xls, I1E1 Logger Data



Test began: 11/30/01 11:00

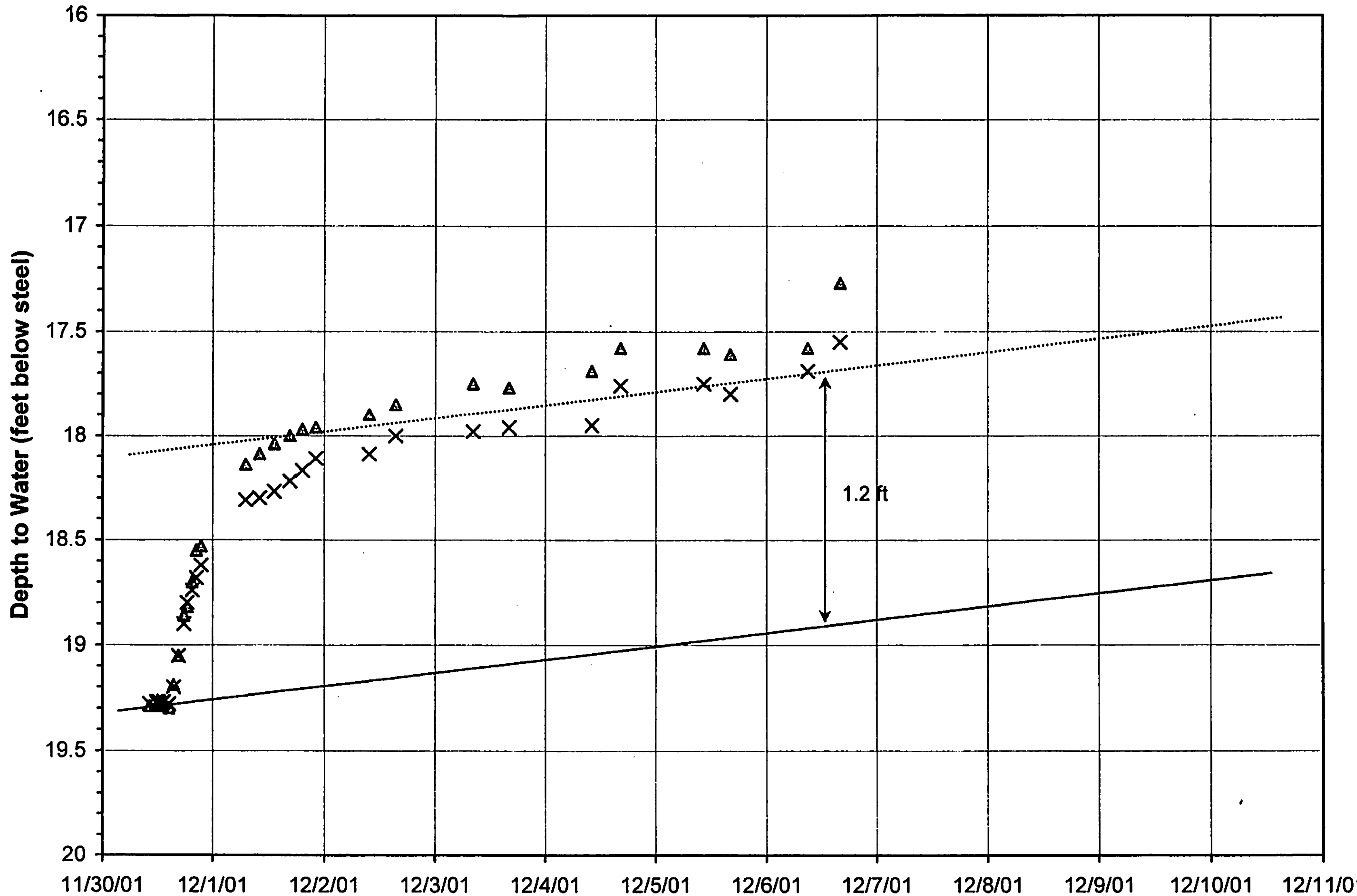
▲ I1/E2S transducer data

◆ I1/E2S Hand Data

FIGURE A2
Water Levels in Piezometer I1/E2

Tulalip Infiltration Tests
I1 Test

Pacific Groundwater Group

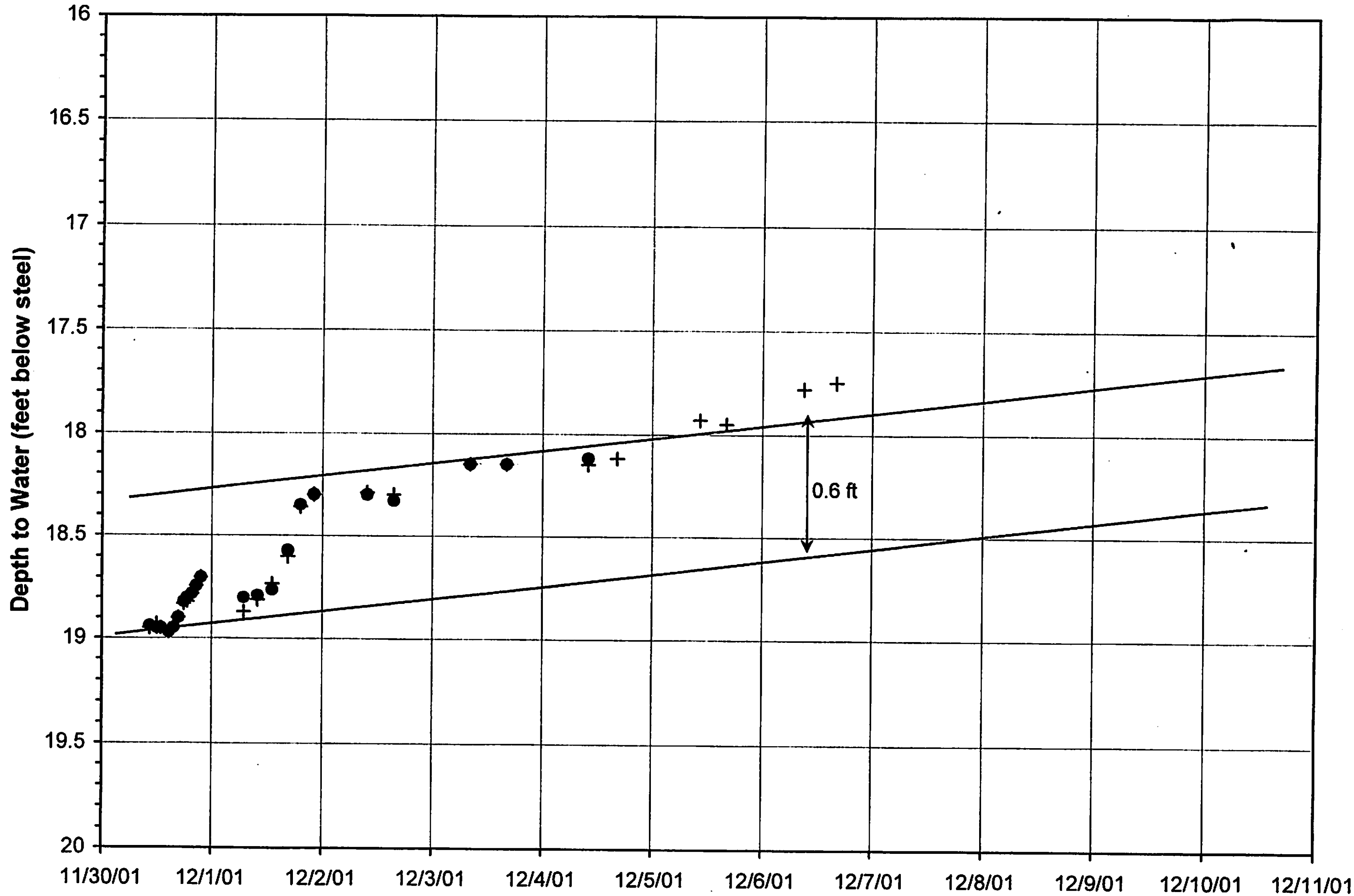


Test began: 11/30/01 11:00

FIGURE A3
Water Levels in I1/E Wells

Tulalip Infiltration Tests
I1 Test

Pacific Groundwater Group



+ I1/E3S

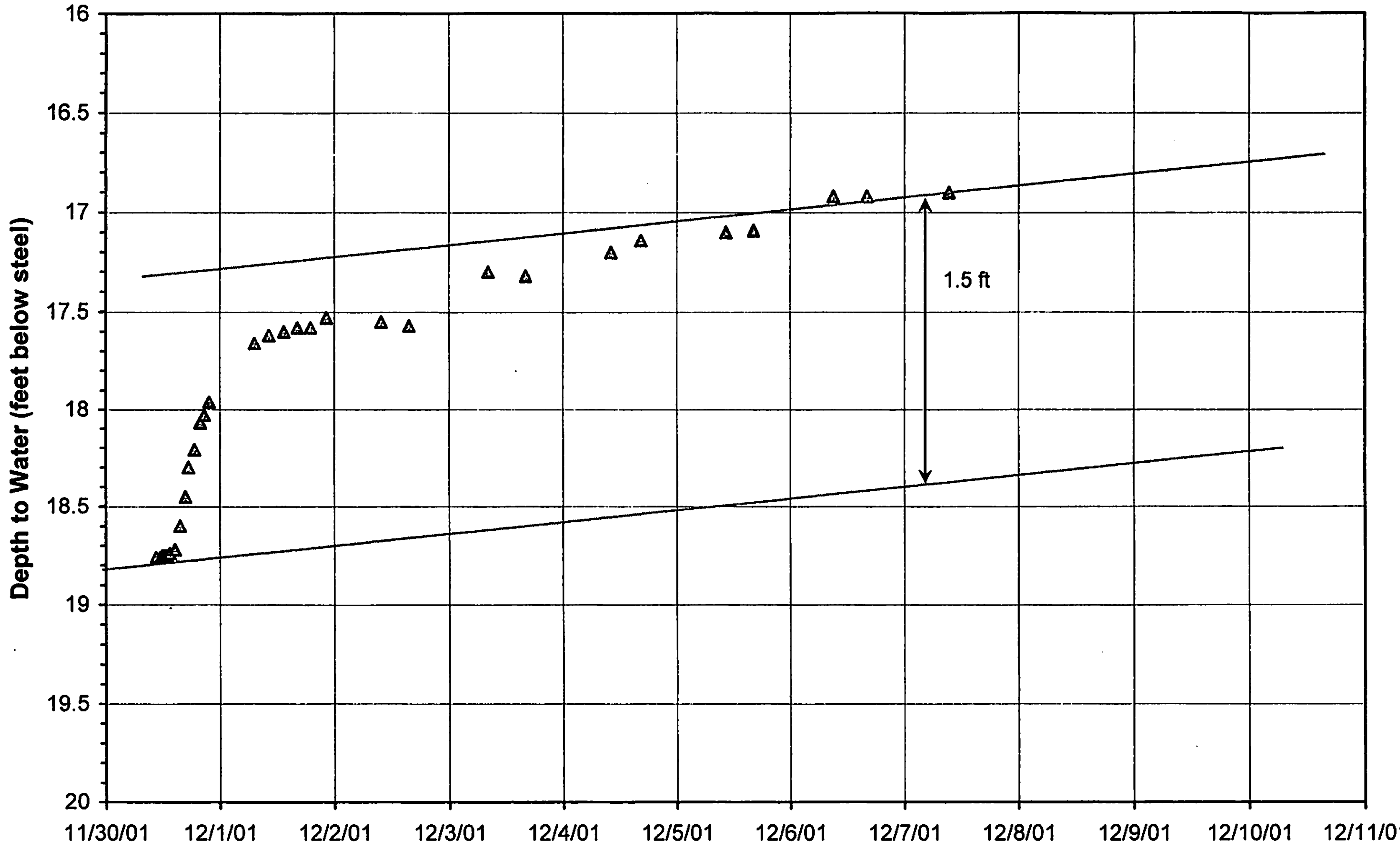
● I1/E3D

Test began: 11/30/01 11:00

FIGURE A4
Water Levels in I1/E Wells

Tulalip Infiltration Tests
I1 Test

Pacific Groundwater Group



I1/S1D

Test began: 11/30/01 11:00

I1/S1S Dry Throughout Test

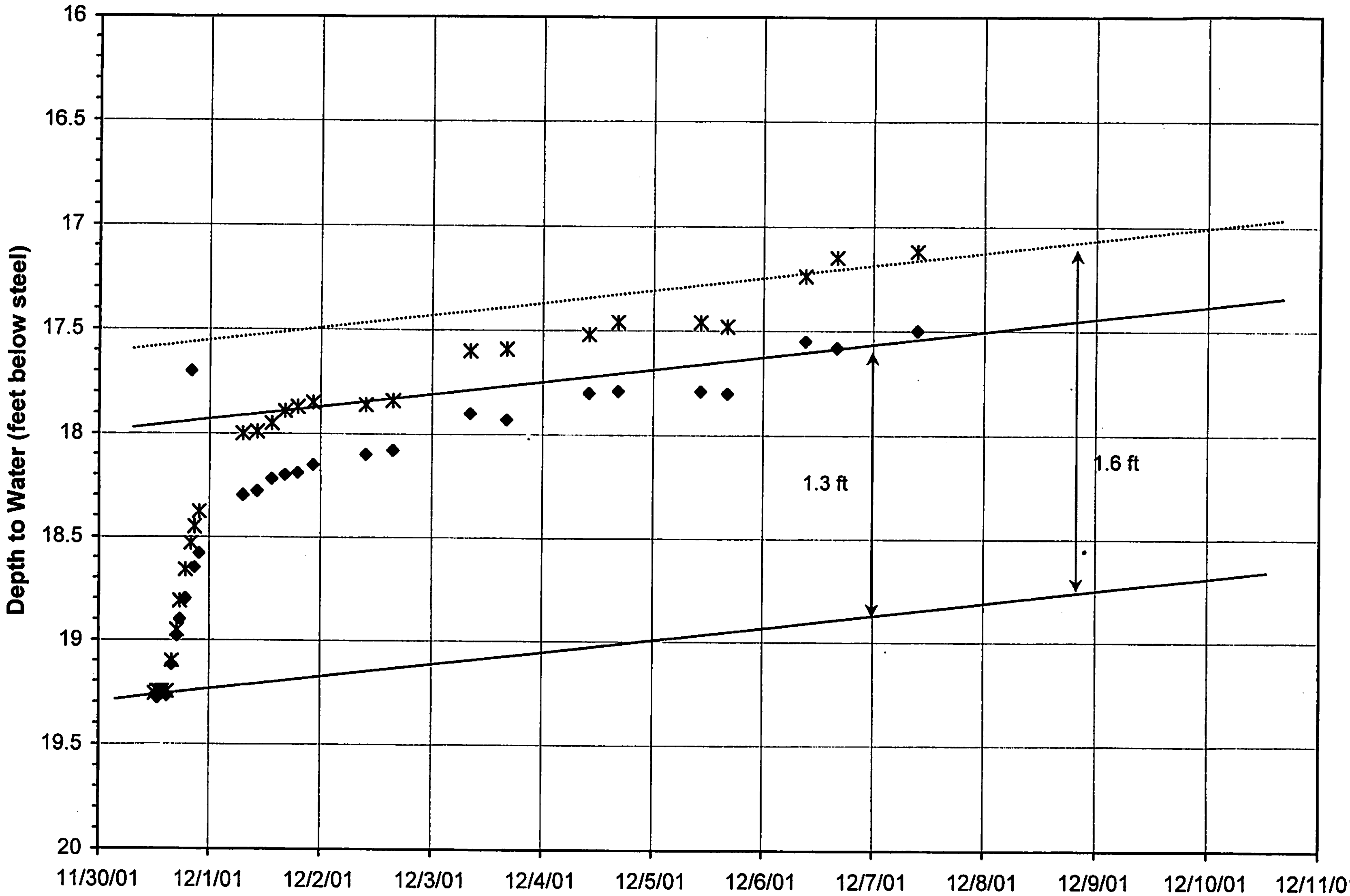
FIGURE A5
Water Levels in I1/S Wells

Tulalip Infiltration Tests
I1 Test

Pacific Groundwater Group

5/13/2002

I1 Arithmetic Plots.xls, I1S1 Plots



I1/S2S

I1/S2D

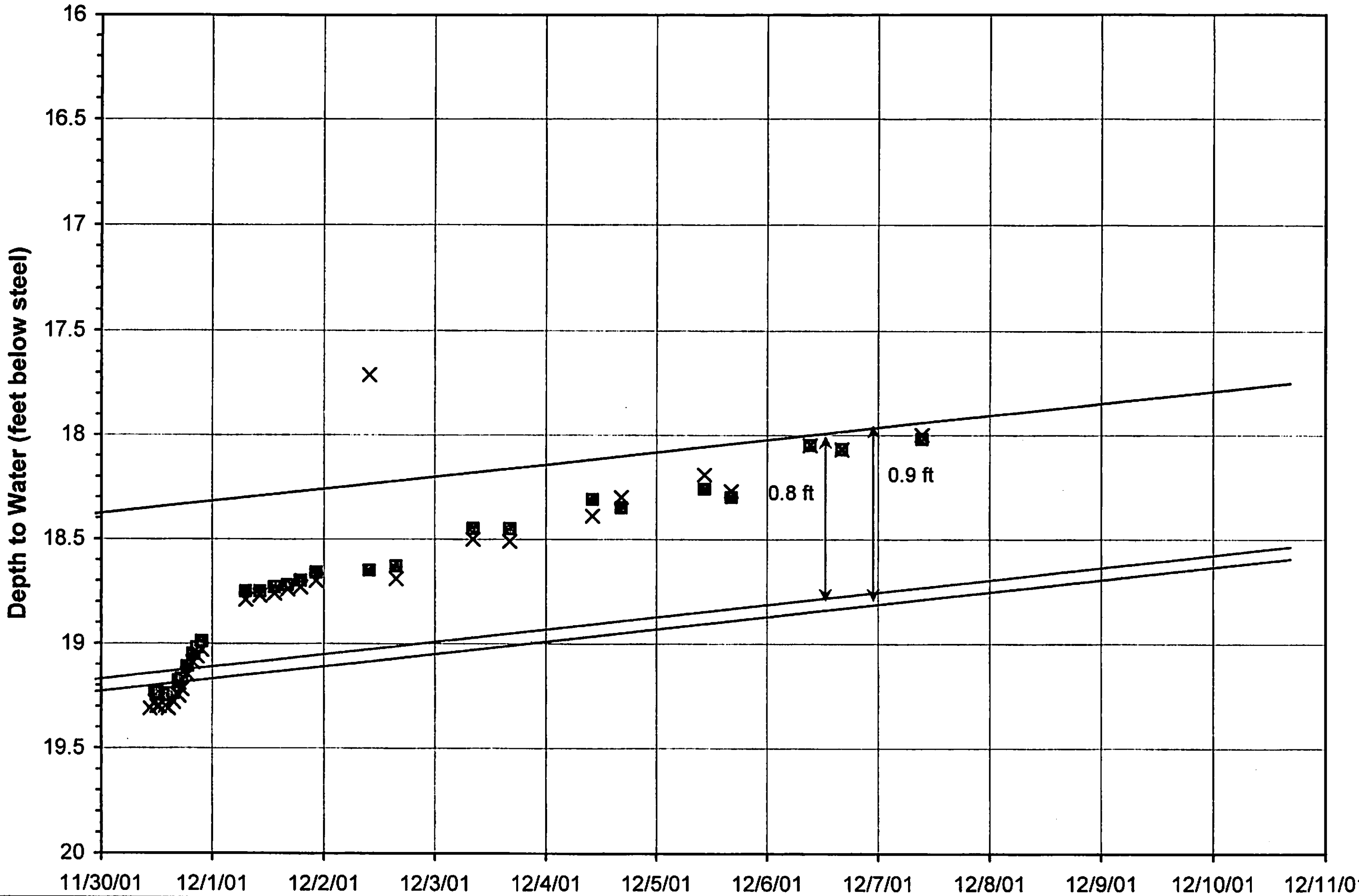
Test began: 11/30/01 11:00

I1/S2S Dry until 63 minutes into test

FIGURE A6
Water Levels in I1/S Wells

Tulalip Infiltration Tests
I1 Test

Pacific Groundwater Group



I1/S3S

I1/S3D

Test began: 11/30/01 11:00

FIGURE A7
Water Levels in I1/S Wells

Tulalip Infiltration Tests
I1 Test

Pacific Groundwater Group

Appendix B

I-2 Local Hydrogeologic Conditions

Boring B-3, advanced by AMEC on November 7 2001, is near infiltration test I-2, and is the best exploration of stratigraphy at that location. B-2 encountered the following materials between ground surface and its total depth of 50.5 feet:

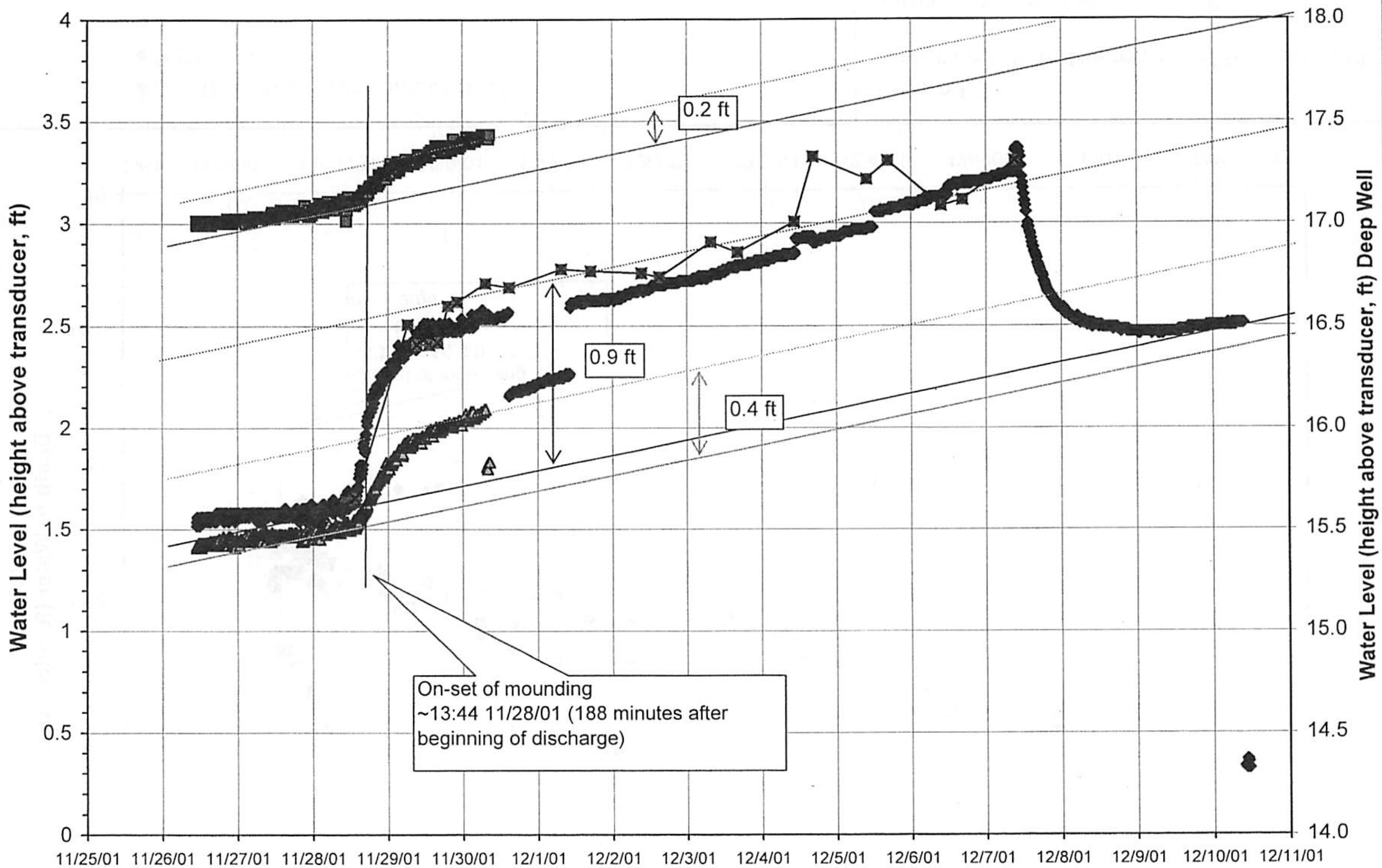
Depth Range	B-2 Material Description
0 to 13.5 feet bgs	Fine to medium SAND, with some coarse sand and trace silt, and a 1-inch silt lens at 12 feet.
13.5 to 15.5 feet bgs	6-inch lens of clayey SILT, underlain by interbedded silty SAND and SAND with silt
15.5 to ~30 feet bgs	Fine to coarse SAND, with trace silt, and some gravel near bottom
~30 feet to 47.5 feet bgs	Fine to medium SAND, fining downward to fine SAND with some silt, includes a 4-inch silty-sand lens at 40 feet, and interbedded fine to medium SAND and silty SAND between 47 and 47.5 feet.
47 feet to at least 50.5 feet	clean sand?

Twelve piezometers were installed in six boreholes surrounding site I-2. Piezometers were installed in boreholes 2.0, 10.1, and 49.7 feet from the east edge of the 12x12-foot infiltration basin. Another set of piezometers was installed in boreholes advanced 1.9, 9.8, and 50.4 feet from the south edge of the basin. Soils were logged by observing soils coming off the auger flights, but no soil samples were collected ahead of the auger. In general, one shallow and one deep piezometer were installed in each borehole. The shallow set of piezometers consist of 2-foot long well screens from 13 to 15 feet depth (**Table 5** of the main text), which are surrounded by coarse sand packs from 10 to 20 feet depth (with one exception). The deeper set of piezometers consist of 2-foot long well screens from 33 to 35 feet depth, surrounded by sand packs from 30 to 35.5 feet depth. Seals between the sand packs are composed of bentonite.

Pre-test water levels in the piezometers indicate that depth to the water table was between 13 and 14 feet bgs and that little vertical flow was occurring (heads in shallow and deep piezometers were similar).

Water was discharged to the site I-2 infiltration test between 10:36 on 11/28/01 and 10:40 on 12/7/01. Discharge rate was a nearly constant 7.8 gpm into the 144 square-foot basin. Mounding was observed in the closest, most frequently monitored piezometer (I2/E1S) about 188 minutes after the beginning of water discharge.

Figures B1, B2, B3, B4, and B5 show water levels in nearby piezometers as the test progressed. **Figure B1** most clearly indicates both the mounding effects, water level recovery after mounding, and the regional water level increases that occurred over the test duration as a result of substantial precipitation. The trend on **Figure B1** was used to identify that water levels increased at an average rate of 0.067 feet per day over the testing period. Solid lines with a slope of 0.067 feet per day were drawn on all I-2 plots, starting at the intersection of the data and the on-set of mounding. Dotted lines with a slope of 0.067 feet per day were drawn through a graphically estimated average steady-state water level after about 2 days of water discharge. Based on visual inspection of the plots, water levels achieved a steady state as of that time. The distance between the two lines is the average steady-state water level increase caused by infiltration. Those mound heights are summarized in **Table 7** of the main text.



Test began: 11/28/01 10:36

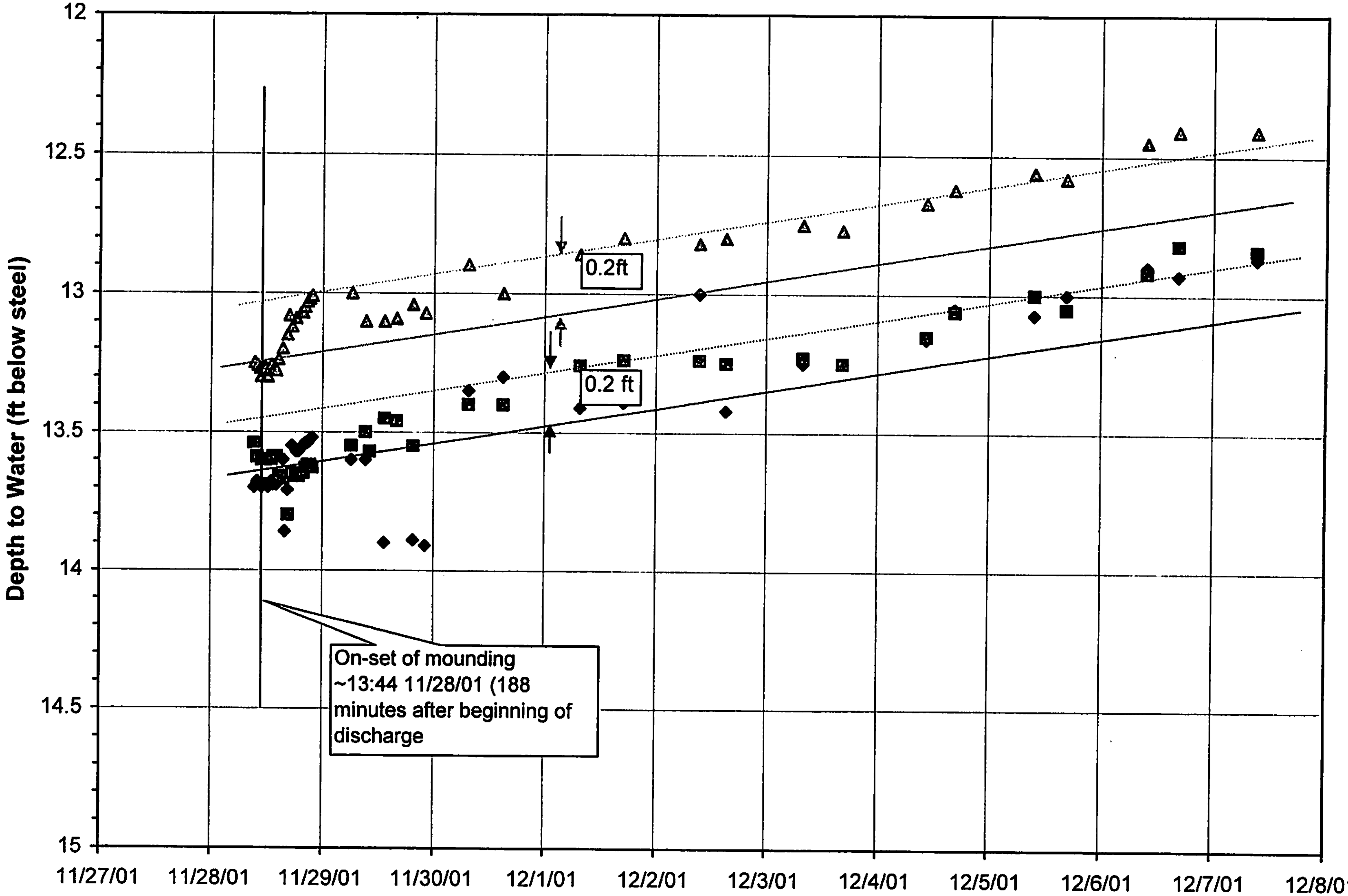
FIGURE B1
Water Levels in Piezometer I2/E1 and I2/E2

Tulalip Infiltration Tests
I2 Test

Pacific Groundwater Group

5/13/2002

I2 Arithmetic Plots.xls, Logger Data



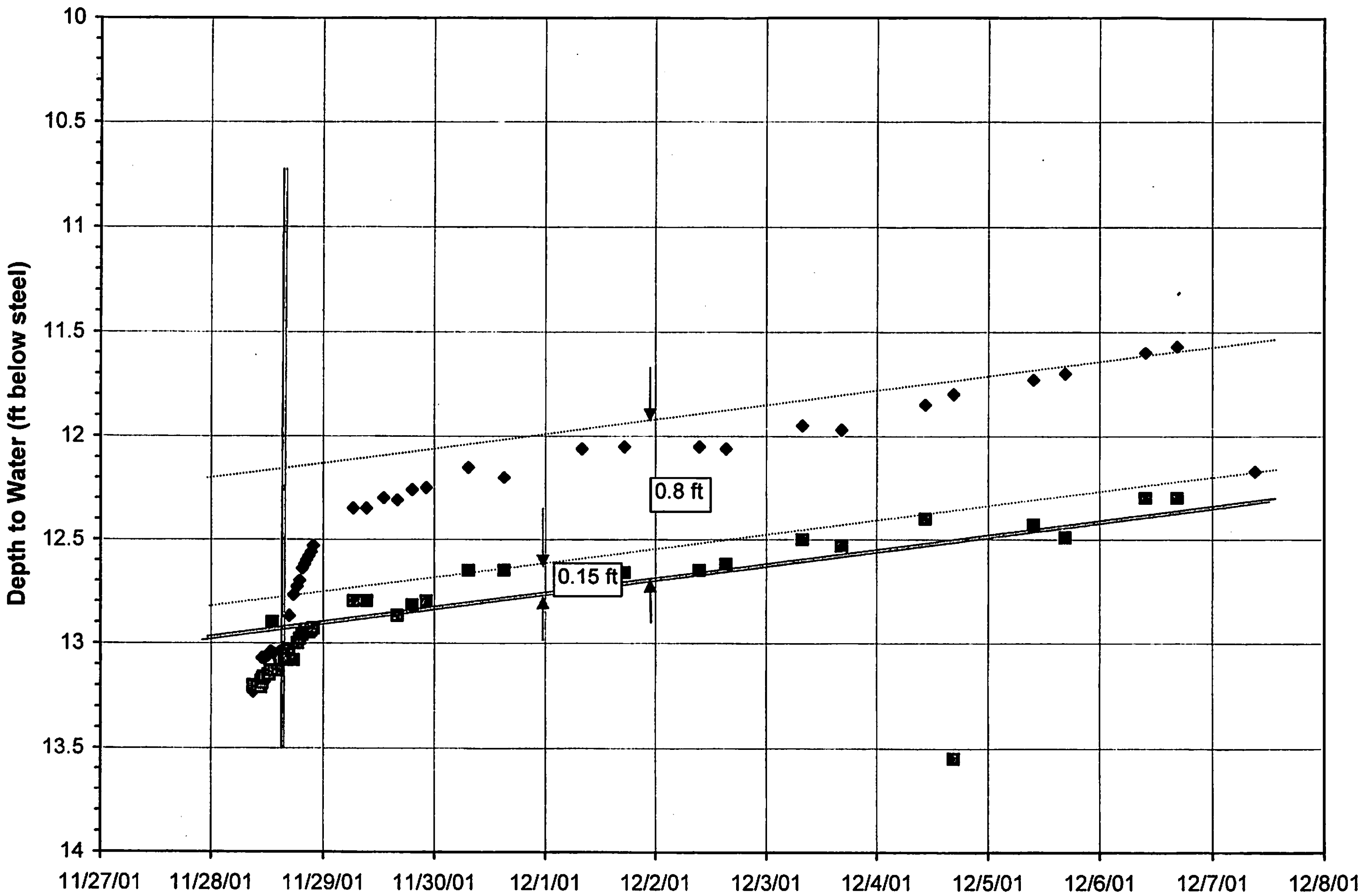
I2/E2D
I2/E3S
I2/E3D

Test began: 11/28/01 10:36

Figure B2
Water Levels in Piezometer I2/E1 and I2/E2

Tulalip Infiltration Tests
I2 Test

Pacific Groundwater Group



◆ I2/S1S

▣ I2/S1D

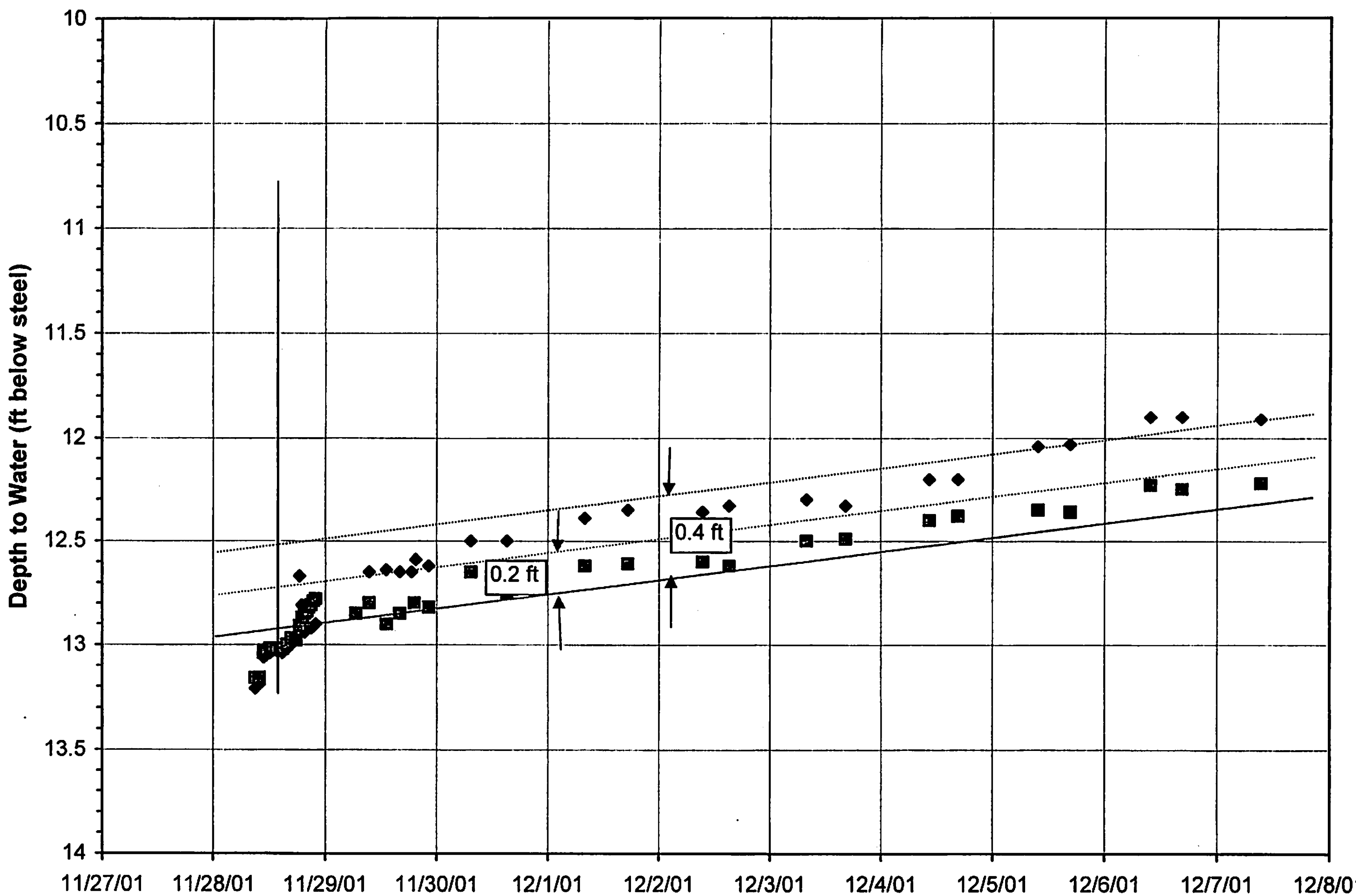
See note on data worksheets

Test began: 11/28/01 10:36

FIGURE B3
Water Levels in Piezometer I2/S1

Tulalip Infiltration Tests
I2 Test

Pacific Groundwater Group



◆ I2/S2S

▣ I2/S2D

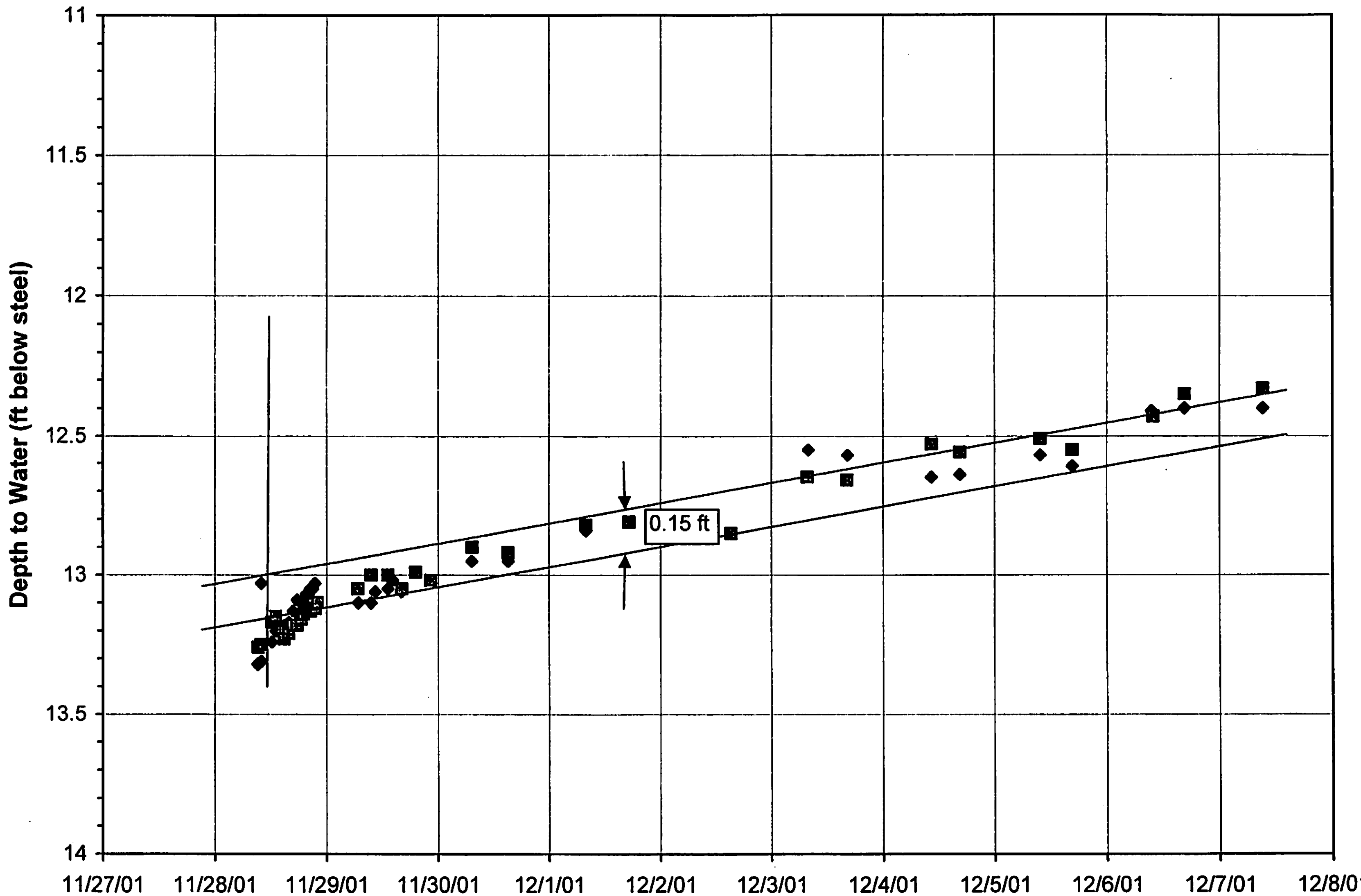
See note on data worksheets

Test began: 11/28/01 10:36

FIGURE B4
Water Levels in Piezometer I2/S2

Tulalip Infiltration Tests
I2 Test

Pacific Groundwater Group



See note on data worksheets

Test began: 11/28/01 10:36

FIGURE B5
Water Levels in Piezometer I2/S3

Tulalip Infiltration Tests
I2 Test

Pacific Groundwater Group